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A MEANS OF PREDICTING SWELLING PRESSURES OF SOILS FOUND IN THE --ETC(U)

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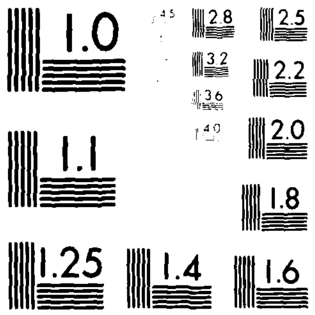

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A [illegible] OF [illegible] SWELLING [illegible] OF SOILS [illegible]
THE ROCKY MOUNTAIN AREA

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Final Report 1[illegible] May 19[illegible]

A report submitted to the Faculty of the Department of Civil, Environmental and Architectural Engineering of the University of Colorado in partial fulfillment of the requirements for the degree of Master of Science, Civil Engineering.

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SWELLING SOILS ARE RESPONSIBLE FOR CONSIDERABLE DAMAGE TO STRUCTURES AND SIGNIFICANT MONETARY LOSSES. THE ABILITY TO PREDICT AND QUANTIFY THIS PHENOMENON HAS BEEN THE BASIS FOR MUCH INVESTIGATION OVER THE LAST 50 YEARS. TO DATE, NO ONE METHOD HAS BEEN FOUND WHICH WILL ADEQUATELY ACCOMPLISH THIS. IN THIS REPORT, THE RESULTS OF HISTORICAL INVESTIGATIONS ARE BRIEFLY SUMMARIZED, AS ARE THE MECHANISMS WHICH ARE INVOLVED IN THE SWELLING PROCESS.

BASED UPON THESE HISTORICAL RESULTS, TWO METHODS FOR EXAMINING EMPIRICAL DATA ARE PROPOSED. THE FIRST METHOD CORRELATES SWELLING PRESSURE TO THE

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NATURAL DRY DENSITY AND LIQUID LIMIT OF THE SOIL. THE SECOND METHOD PARALLELS THE FIRST, EXCEPT THAT THE CORRELATION IS ACCOMPLISHED USING AN INTRODUCED PARAMETER (PLASTICITY INDEX / PERCENT OF SOIL PASSING THE #200 SIEVE) INSTEAD OF THE LIQUID LIMIT. AN EMPIRICAL DATA BASE FROM THE ROCKY MOUNTAIN GEOGRAPHICAL AREA IS EXAMINED USING THESE TWO METHODS. THE DATA BASE CONSISTS OF BOTH CLAY SOILS AND SEDIMENTARY CLAYSTONES. PREDICTIVE EQUATIONS ARE DEDUCED FOR EACH ANALYSIS METHOD. THE FIRST METHOD (USING THE LIQUID LIMIT) PRODUCES BETTER RESULTS, AND COMPARISONS OF PREDICTED VERSUS MEASURED VALUES ARE PRESENTED FOR THIS METHOD. A LISTING OF THE DATA USED IN THE ANALYSES IS INCLUDED IN THE REPORT.

9 Final rept.,

6 A MEANS OF PREDICTING SWELLING PRESSURES
OF SOILS FOUND IN THE
ROCKY MOUNTAIN AREA.

by

16 Walter Ernest Heinz

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Walter Ernest Heinz

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by

Hon-Yim Ko

Date May 19, 1980

Heinz, Walter Ernest (M.S.. Civil Engineering)

A Means of Predicting Swelling Pressures of Soils Found in the Rocky Mountain Area

Report directed by Professor Hon-Yim Ko

Swelling soils are responsible for considerable damage to structures and significant monetary losses. The ability to predict and quantify this phenomenon has been the basis for much investigation over the last 50 years. To date, no one method has been found which will adequately accomplish this. In this report, the results of historical investigations are briefly summarized, as are the mechanisms which are involved in the swelling process.

Based upon these historical results, two methods for examining empirical data are proposed. The first method correlates the swelling pressure to the natural dry density and the liquid limit of the soil. The second method parallels the first, except that the correlation is accomplished using an introduced parameter (plasticity index/percent of soil passing the #200 sieve) instead of the liquid limit. An empirical data base from the Rocky Mountain geographical area is examined using these two methods. The data base consists of both clay soils and sedimentary claystones. Predictive equations are deduced for each analysis method. The first method (using the

III

liquid limit) produces better results, and comparisons of predicted versus measured values are presented for this method. A listing of the data used in the analyses is included in this report.

This abstract is approved as to form and content.

Signed Hungmi Ko
Faculty member in charge of report

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To Dr. Hon-Yim Ko of the Department of Civil and Environmental Engineering, the author extends his heartfelt appreciation for his valuable guidance during the past year. His suggestions and criticisms were instrumental in the preparation of this report.

The author also extends his thanks to two other faculty members, Dr. Robert L. Schiffman, and Dr. Richard H. Atkinson for their participation as Committee Members and their thought-provoking questions.

Notes of thanks are extended to Mrs. Florence Petersen and Mrs. Marie Kindgren. Mrs. Petersen's administrative knowledge was of great assistance, and Mrs. Kindgren rendered diligent service in the preparation of the manuscript.

It is with a very special and deep gratitude that the patience, encouragement and understanding of my wife, Linda, are acknowledged. Her moral support was essential to the completion of this report.

Finally, the author would like to thank his parents, Emily M. Heinz and Ernest Heinz Jr., for many years of love, inspiration and guidance. Their contribution to this report is not quantifiable. They have had however, a profound and pervading influence, for which the author will forever be thankful.

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CHAPTER I

INTRODUCTION

Expansive soils are found throughout the world, and the damage caused to structures which are founded on them is equally universal as well as costly. It has been estimated that the damage caused annually by swelling soils exceeded 2¼ billion dollars in 1973 [11].[1] Damages caused by swelling soils cover a wide range of magnitude varying from purely cosmetic defects such as hairline cracking to major structural distresses. For example, cases of interior building walls bowing by up to 12 inches and pier uplift of some 4 inches have been recorded.[2] Facts and figures of this magnitude are staggering and underscore the need for a more thorough understanding of this phenomenon. When one considers that the future will bring with it an ever increasing need to utilize less desirable and previously avoided locations for construction, the problem becomes even more meaningful.

The problems caused by expansive soils were first recognized in the late 1930's and thus this is a

[1] Numbers inside brackets indicate bibliography reference.

[2] Taken from files of Chen and Associates, Inc., Consulting Soil Engineers, Denver, Colorado.

relatively new area in soil mechanics. Significant research in this area has been done throughout the world; however, the lack of standardized testing procedures used in the research examining the expansiveness of soil has resulted in poor correlation of test data. For example, "percent swell" data cannot be readily correlated as confining pressures used vary from experiment to experiment.

The testing which has been done to date and the conclusions which have been drawn cover both investigative techniques and the examination of empirical data. Chen [3], and Seed, Woodward and Lundgren [22] are among those who have conducted the former, while Holtz and Gibbs [10] and Vijayvergiya and Ghazzaly [27] are among the many who have done the latter.

It is not the purpose of this report to establish any standardized testing procedures. Instead, it is aimed at providing an empirically derived means for predicting the swelling pressure of expansive soils found generally in the Rocky Mountain area of the United States. The bulk of test samples which will be examined are from Colorado. The data analysis used combines conclusions drawn by Chen [3], Seed, Woodward and Lundgren [22], and Vijayvergiya and Ghazzaly [27].

CHAPTER II

NATURE OF EXPANSIVE SOILS

A. Distribution of Expansive Soils

As alluded to in the introduction, expansive type soils are to be found throughout the world. As of 1969, the list of countries which are known to contain expansive soils includes [6]:

Argentina	Iran
Australia	Mexico
Burma	Morocco
Canada	Rhodesia
Cuba	South Africa
Ethiopia	Spain
Ghana	Turkey
India	U.S.A.
Israel	Venezuela

The exact locations of these deposits within these countries show that expansive soils are generally to be found in semi-arid regions, or in those areas where average annual evapo-transpiration exceeds precipitation (Fig. 1). The repetitive drying and wetting cycles which occur in such regions contribute to the swelling of such soils. All soils located in such regions do not exhibit expansive characteristics, but those that do, have additional characteristics which will be further explained below.

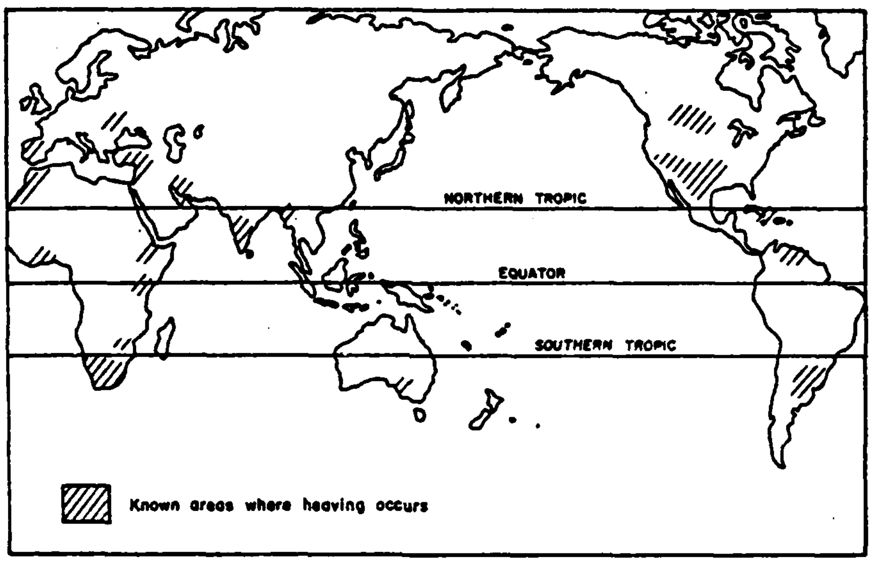


Figure 1

Distribution of Reported Instances
of Heaving [6]

The study of expansive soils is in a relative infancy stage as compared to soil mechanics. The expansive properties of soil did not begin to be recognized until the 1930's. During this period in the United States, the expansion of the population into areas containing expansive soils, coupled with the more widespread use of brick in the construction of residential type structures, served to highlight the damaging effects of expansive soils. Predominantly wooden structures were constructed prior to this period, and since these were more flexible, they tended to absorb the stresses caused by expanding soils. The more rigid brick structures, on the other hand, tended to show visibly the effects of expanding soils in the form of cracks. Initially, such cracking was "explained" as differential settlement and poor construction, and it took several more years before such damage was recognized as being attributable, at least in part, to expansive soils.

In light of the above, it is reasonable to expect that the presence of expansive soils is not limited to the countries listed above, but that additional regions exist and will present similar problems as buildings are erected in as yet, unoccupied regions.

B. Mineralogy

Soils can be categorized as cohesive and non-cohesive (cohesionless) as one means of describing behavioral characteristics. The cohesionless soils tend to be comprised of bulky particles while the cohesive soils tend to contain smaller, flat, plate-like particles that are also known as clay particles. Clays are further divided into three major groups, kaolinites, illites and montmorillonites, based upon their molecular structure. Research has shown that expansive soils tend to be high in clay content and especially in montmorillonite content.

Research has shown all clay minerals to be predominantly crystalline in nature. Two basic structural "building blocks" are found to predominate in the three major clay minerals. These basic units are the silica tetrahedron, and the octahedral aluminum hydroxide. The silica tetrahedron consists of a silicon atom which is surrounded by four oxygen atoms which are located at the apexes of equilateral triangles (Fig. 2a). These units may combine as shown in Fig. 2b such that the base plane is comprised of oxygen atoms arranged in a hexagonal pattern, with adjacent tetrahedra sharing oxygen atoms. The silicon atoms may be oriented such that a plane of these atoms exists, and a sheet-like particle is the result. The octahedral aluminum hydroxide element consists of a central aluminum atom which is surrounded by both oxygen

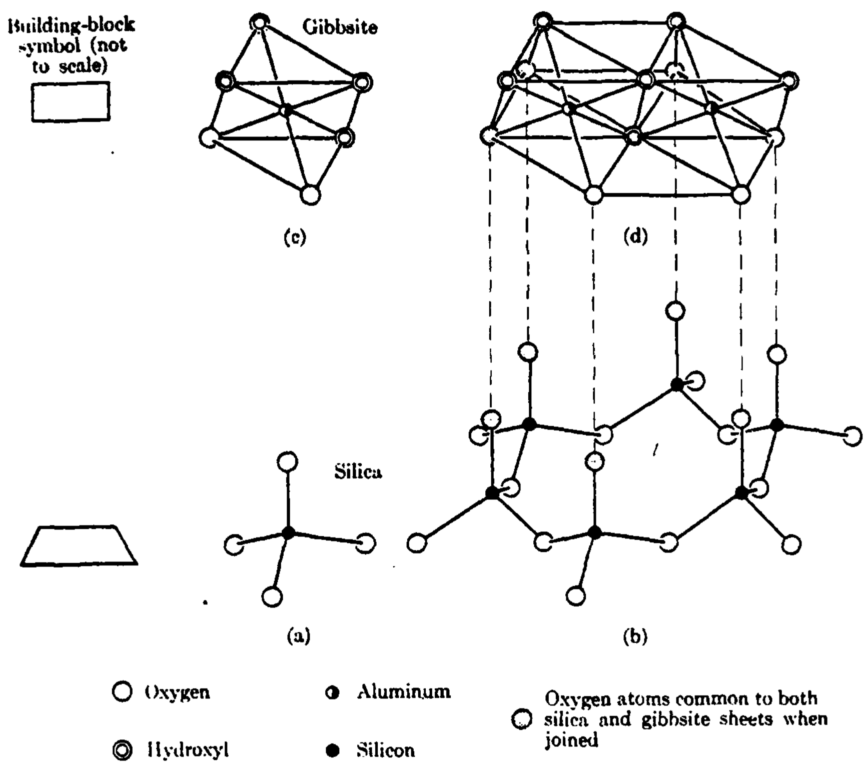


Figure 2

Molecular Structure of Silica and Gibbsite Sheets [23]

(O^{2-}) and hydroxyl ($(OH)^-$)ions, and is commonly called hydrated alumina (Fig. 2c). It can combine as shown in Fig. 2d, also producing sheet-like particles known as Gibbsite. Figure 2 also shows schematic representations of these building blocks which will be utilized below to depict the structure of the three major clay minerals.

The three major clay mineral groups are named after their predominant mineral, although they include other minerals as well (Table 1). The kaolinites are typified by a combination of the two basic building blocks such as shown schematically in Figure 3. This configuration produces an electrically neutral sheet of the mineral kaolin as the unsatisfied oxygens of the silicon tetrahedra are shared by the hydrated alumina sheet (see Fig. 2b and d). Continued stacking of these units is possible, however, kaolin normally occurs as a particle of .05 micron in thickness, and from 0.5 to 1.0 micron in diameter. The relative magnitude of bond strengths existing in kaolin particles is also shown in Fig. 3. Variations in stacking arrangements between these building blocks result in the other clay minerals of the kaolinite group.

The basic structural element of montmorillonites is comprised of two silica tetrahedra sheets separated by a hydrated alumina sheet. The stacking pattern of such

Table 1

CLAY MINERALS [23]

I. *Kaolin group*

1. Kaolinite $Al_4Si_4O_{10}(OH)_8$
2. Dickite $Al_4Si_4O_{10}(OH)_8$ } Rare
3. Nacrite $Al_4Si_4O_{10}(OH)_8$ } Rare
4. Halloysite, also known as metahalloysite, or halloysite ($2H_2O$) nonplastic:

   $$Al_4Si_4O_{10}(OH)_8$$

5. Endellite, also known as halloysite, hydrated halloysite, or halloysite ($4H_2O$) nonplastic:

   $$Al_4Si_4O_{10}(OH)_8 \cdot 4H_2O$$

   Note: Halloysite ($4H_2O$) loses water between site and laboratory at moderate temperatures, to form metahalloysite ($2H_2O$) with different engineering properties.

6. Allophane—amorphous silica aluminum mixture
7. Anauxite

II. *Montmorillonite group (interlayer water molecules omitted)*

1. Montmorillonite $(Al_{1.67}\overset{\overset{Na_{0.33}}{\uparrow}}{Mg_{0.33}})Si_4O_{10}(OH)_2$
2. Beidellite $Al_{2.17}(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_{0.33}}Si_{3.17})O_{10}(OH)_2$

   or

3. Beidellite $Al_{2.22}(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_1}Si_3)O_{10}(OH)_2$
4. Nontronite $Fe_{2.00}(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_{0.33}}Si_{3.67})O_{10}(OH)_2$
5. Nontronite $Fe_{2.17}(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_{0.33}}Si_{3.17})O_{10}(OH)_2$

   or

6. Nontronite (aluminian) $Al_{2.22}(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_1}Si_3)O_{10}(OH)_2$
7. Hectorite $(Mg_{2.67}\overset{\overset{Na_{0.33}}{\uparrow}}{Li_{0.33}})Si_4O_{10}(F, OH)_2$
8. Saponite $Mg_3(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_{0.33}}Si_{3.67})O_{10}(OH)_2$
9. Saponite (aluminian) $(Mg_{2.67}Al_{0.33})(\overset{\overset{Na_{0.33}}{\uparrow}}{Al_{0.67}}Si_{3.33})O_{10}(OH)_2$
10. Sauconite $(Si_{3.47}\overset{\overset{Na_{0.33}}{\uparrow}}{Al_{0.53}})(Al_{0.22}Fe_{0.17}Mg_{0.18}Zn_{2.40})O_{10}(OH)_2$
11. Talc $Mg_3Si_4O_{10}(OH)_2$
12. Pyrophyllite $Al_2Si_4O_{10}(OH)_2$

III. *Illite group (amount of K present varies)*

$$K_y(Al_4Fe_4Mg_4Mg_6)(Si_{8-y}Al_y)O_{20}(OH)_4$$

IV. *Miscellaneous minerals*

1. Attapulgite $Mg_5Si_8O_{20}(OH)_2 \cdot 8H_2O$
2. Sepiolite (meerschaum) $H_4Mg_2Si_3O_{10}$
3. Sericite
4. Mixed layer aggregates
5. Vermiculite
6. Glauconite
7. Chlorite
8. Diaspore

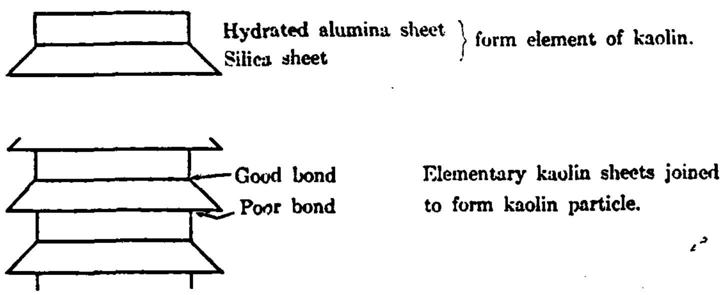


Figure 3

Structure of Kaolin Particle [23]

montmorillonite sheets, as shown in Fig. 4a, forms the montmorillonite particles. As can be seen, the bond between the three-layered montmorillonite sheets is a very weak bond, and therefore water molecules (which are of the right size) may easily enter in the area of this bond. Up to six layers of water molecules may enter at these locations and thus the montmorillonites have high swell potential. Typically montmorillonite particles exist in very small particle size of about .05 micron in diameter and with a diameter to thickness ratio of up to 400:1 [23]. Illites are similar in composition to montmorillonites except that in the region of silica tetrahedra base planes, potassium ions exist (Fig. 4b).

Comparatively, the potassium-oxygen bonds are stronger than the oxygen-oxygen bonds present in the montmorillonites and thus the lattice is less susceptable to separation and infiltration of water molecules. The result is that illites swell less than montmorillonites. Typically illite particles have diameters of 0.05 microns and a diameter to thickness ratio of 50:1 [23].

C. <u>Mechanics of Swelling</u>

Clay particles exhibit concentrations of electric charge around their surface with this charge usually negative. Because of this, water molecules (which are bipolar) orient themselves around the particle surface

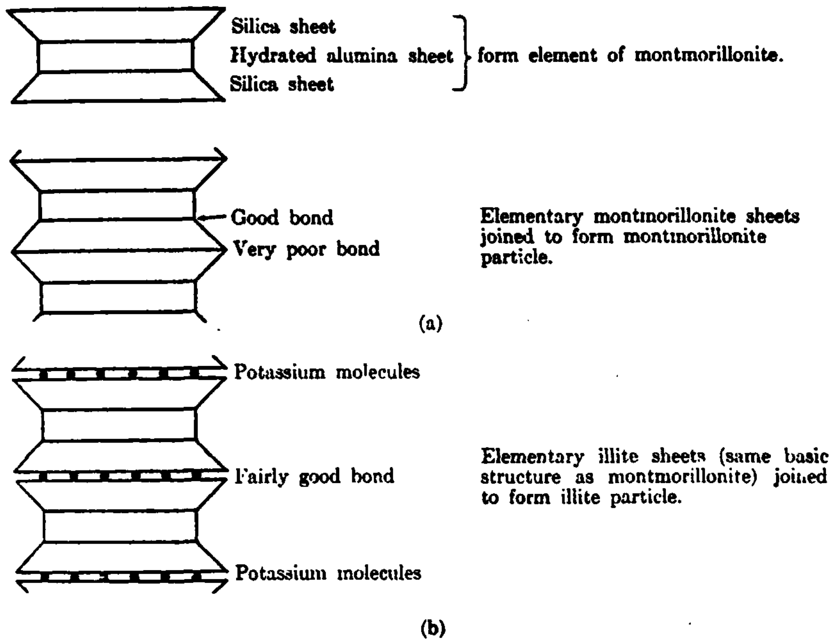


Figure 4

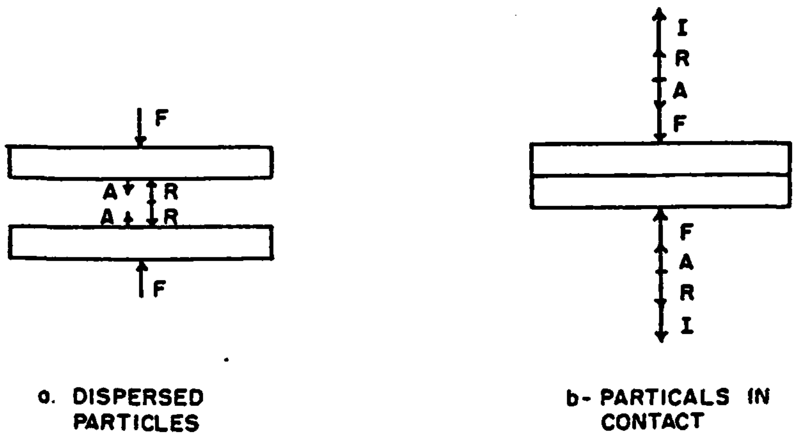
Structure of Montmorillonite and Illite Clay Particles [23] and form an encircling layer of molecular water known as the double layer. The further away from the surface of the particle a molecule of water is, the less is the attractive force acting on it. Clay particles are separated from each other by these films of adsorbed water and the thicker such separating layers are, the more easily some of this water may be expelled from the soil. The water closest to the clay particle is held tightly and for all practical purposes, acts as a solid. The more remotely that water is located from the particle, the less viscous it behaves and the more freely it can be lost. Expulsion of this water may result, for example, from an increase in surcharge pressure on the soil or from evapotranspiration. Regardless of the process, the soil can "shrink" resulting in shrinkage cracking or dessication. This shrinkage is due to compressive stresses transferred to the soil from the menisci of water remaining between clay particles. This is analogous to the compressive forces which occur in a capillary tube due to the rise of water under the action of surface tension (see [18] for a more detailed explanation). Similarly, once such water is expelled, a deficiency exists in the double layer and the clay then has an affinity, or thirst, for water. Thus, when moisture becomes available, it is readily taken in, sating this deficiency and enlarging the double layer.

As double layers enlarge and interact, repulsive electrostatic forces arise and expansion of the overall soil mass is the result [15].

The above explanation is one of several theories which seek to explain the expansion process. Another, proposed by Lambe and Whitman, explains the process of expansion in terms of the effective stress equation [16]. Although these individuals express serious reservations as to the overall validity of "forcing" the expansion process to fit the effective stress equation, they suggest that for clays which undergo expansion (as a result of either contact with water or the removal of an effective stress) the interparticle repulsive pressure R exceeds the interparticle attractive pressure A. Their doubts are based upon:

1) whether the only net force transmitted between adjacent particles is that derived from externally applied loads
2) whether all forces carried by the soil mineral skeleton are transferred through the soil contact area
3) whether all pore water pressure is transmitted by the water area.

(For a more detailed explanation, see reference [16].) For the case of dispersed clay particles completely separated by the double layer (Fig. 5): $\sigma = \bar{\sigma} + u = R - A$.



F = EXTERNALLY DERIVED FORCE
A = ELECTRICAL ATTRACTION
R = ELECTRICAL REPULSION
I = CONTACT INTERACTION

Figure 5

Forces Between Adjacent Particles [16]

This net pressure requires some value of effective stress to counteract it, if a state of zero volume change is to exist. Expansion can occur when this state of equilibrium is altered, be it the result of a decrease in the effective stress, or an increase in the net repulsive pressure, (R - A). This latter condition can occur if additional water becomes available to the clay, for example.

Still another explanation of expansion in clays considers the difference in osmotic pressure between the double layer surrounding the clay particles and the free water which surrounds it. Due to the preferred electrical orientation of the double layer, cations are present in it to a higher degree than in surrounding water. The effect of these concentrations of cations is to allow them to function as a membrane, permeable to the flow of water but impermeable to the flow of cations. The effect that this has is to create a differential osmotic pressure between the double layer and the surrounding water. It is this pressure differential which causes a repulsive force between particles, and the thicker the double layer is, the greater this becomes. As soil particles become smaller in size, their amount of surface area increases per unit volume, and the volume of the double layer increases proportionately. Also the type cations present in the double layer can influence the size of the double layer. Monovalent cations are able to exist at a greater distance from clay particles in a given concentration than are multivalent cations. Since larger double layers have greater capacity to absorb water, soils with monovalent double layers exhibit higher swelling potential. The combination of the two above conditions, smallness of particle size and cation concentration of the double layer, can result in a very high affinity for water and large amounts of swelling. Montmorillonites are the smallest in size of the clay minerals (Table 2), and generally these soils are more prone to swelling than the kaolinites or illites. Likewise, monovalent montmorillonites, such as sodium montmorillonite have even greater expansive properties.

The above theories regarding expansion in clays explain the phenomenon from different viewpoints. Research data exists to support each one and thus the question of which one controls the expansion of clay soils cannot be answered. Rather, as each has been shown to have an effect, it is reasonable to assume that each plays a part in the process. Physical conditions may cause one or the other to predominate in any given situation, but all seem to play a role in the process to some degree. How much so is beyond the intent of this report. The purpose of the foregoing has been to briefly review theories which explain the mechanics of the expansion process.

Table 2

CHARACTERISTIC VALUE RANGES FOR PHYSICAL PROPERTIES OF CLAY MINERALS [3]

	Kaolinite	Illite	Montmorillonite
Particle thickness	0.5 – 2 microns	0.003 – 0.1 microns	Less than 9.5 A
Particle diameter	0.5 – 4 microns	0.5 – 10 microns	0.05 – 10 microns
Specific surface (sq. meter/gram)	10 – 20	65 – 180	50 – 840
Cation exchange capacity (milliequivalents per 100g)	3 – 15	10 – 40	70 – 80

(After Woodward-Clyde & Associates, 1967)

CHAPTER III

TESTS FOR THE IDENTIFICATION OF EXPANSIVE SOILS

As can be imagined from the discussion in the preceding chapter, there are many factors which influence the swelling of clay soils. The attempts to find means of identifying such soils have been numerous and have involved the entire range of investigative tools available in present day technology. Basically these attempts have fallen into two distinct categories : those methods which seek to examine mineralogically expansive clays and those methods which attempt to relate the phenomenon of expansion to either volume change characteristics or physical properties of such soils. The former category has used techniques such as: 1) microscopic examination using the electron microscope, 2) X-ray diffraction, 3) differential thermal analysis, 4) chemical analysis and 5) dye adsorption analysis. The latter category (which will be discussed in the next chapter) has sought to relate volume changes or potential volume changes to: 1) free swell testing, 2) the Atterberg limits of the soils, 3) colloid content, 4) linear shrinkage and

5) mechanical measurement. The methods of each of these categories will be discussed, however, the former category will receive less attention as it employs exotic, research oriented techniques which are outside of the realm of the practicing civil engineer. Additionally, although these techniques provide an understanding of the causes of expansion, they do not provide the quantitative data necessary to predict expansion potential.

The first three of the techniques in the first category (microscopic, X-ray and thermal) have proven to be the most successful, however, no single technique is completely reliable, especially when more than one clay mineral is present in the soil. Judicious combining of these techniques is often necessary to obtain valid results.

A. Electron Microscope

The advantage that the electron microscope has over a stereoscopic microscope is that it can identify visually the small clay particles present in expansive clays. This is of great value in that seeing such particles reduces the interpretative judgment which would be required were only other techniques available. Research has shown that two clays may produce very similar results when analysed by other means (e.g.

X-ray diffraction or differential thermal analysis) and yet when viewed with the electron microscope, differences are clearly discernable. The establishment of such differences sheds a new dimension on the problem and allows investigation of their significance. The mineralogic composition, texture and internal structure are readily revealed by using the electron microscope. Research has already shown montmorillonites to be very fine, wavy particles (Figure 6) while non-swelling clays appear as flat, comparatively thicker plates. A typical electron microscope picture of kaolinite is shown in Figure 7.

B. X-ray Diffraction

X-ray diffraction has proven to be a very satisfactory technique in the identification of clay minerals, and to date is the most reliable means for evaluating clay mixtures. This technique compares the ratio of intensities of diffraction lines caused by the impinging of X-rays on the examined substance to that caused by a standard substance. Research has also been done on expansive soils whereby the spacings between clay particles of an expanded soil have been measured as well as the variation that occurs in these spacings for different degrees of swelling. Also, the changes that adsorption of water causes on these spacings have also been monitored using the X-ray diffraction technique.

Figure 6
Electron photomicrograph of montmorillonite (bentonite). Picture width is 7.5 μm [19]

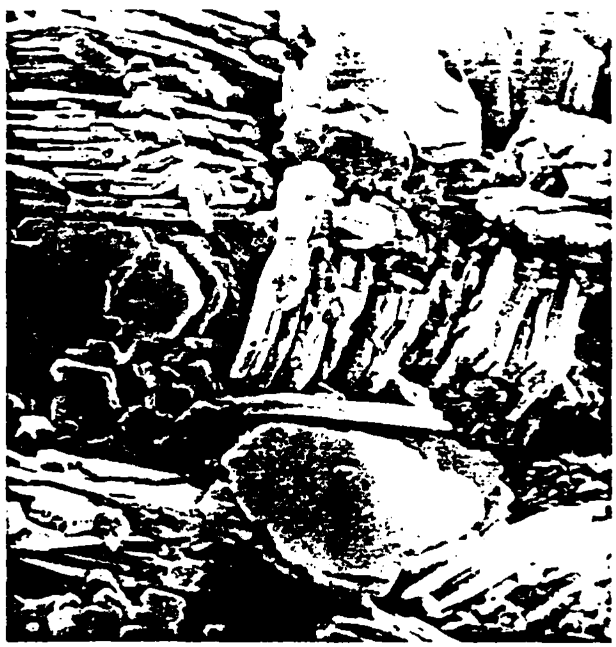


Figure 7
Electron photomicrograph of well-crystalized kaolinite. Picture width is 17 μm [19]

C. Differential Thermal Analysis

Differential thermal analysis is a technique which simultaneously heats both a test sample and a thermally inert material at a constant rate to a high temperature (> 1000°C) while continuously monitoring temperature differences between the two substances. The resulting data is plotted (ΔT vs T). This plot, known as a thermogram, is then compared to known thermograms. Similarities indicate the presence of the known material. Differential thermal analyses on expansive soils have proven not to be very definitive by themselves, however, when used in conjunction with other techniques such as X-ray diffraction or chemical analysis, this method becomes a more valuable tool.

D. Dye Adsorption

Dye adsorption employs dyes or chemical reagents which will display characteristic colors when adsorbed to identify the presence of the clay minerals. Pretreating a clay sample with acid, for instance, will cause various colors to be exhibited when the dye is adsorbed by the clay. The color which manifests itself is dependent upon the base exchange capacity of the clay mineral(s) which exist in the sample. Montmorillonites may be so detected in concentrations as low as 5%. However, this technique is also most often used in conjunction with the other tests, since its reliability is not universally accepted. A variation of this technique is to measure the quantity or rate of adsorption of ethelyne glycol and glycerol by clay minerals. Since adsorption is related to the specific surface area of the mineral, montmorillonites adsorb proportionately more, as would be expected.

Chemical analysis is also a technique which is seldom used alone, since it can be very effective when examining individual clay minerals, but loses reliability when a mixture of clay minerals is involved. Chemical analysis entails determining the amounts of various chemical molecules which comprise clay minerals such as kaolinite, illite, or montmorillonite. Tables 3 and 4 show typical test results of such analyses [8].

Table 3

CHEMICAL ANALYSES OF KAOLINITE MINERALS [8]

	1	2	3	4	5	6	7
SiO_2	46.90	44.81	45.20	46.77	44.59	54.32	48.80
Al_2O_3	37.40	37.82	37.02	37.79	36.83	29.96	35.18
Fe_2O_3	0.65	0.92	0.27	0.45	1.14	2.00	1.24
FeO	......	....	0.06	0.11			
MgO	0.27	0.35	0.47	0.24	0.39	0.14	
CaO	0.29	0.43	0.52	0.13	1.02	0.32	0.22
K_2O	0.84	....	0.49	1.49	0.32	......	0.40
Na_2O	0.44	....	0.36	0.05	0.13	0.37	0.25
TiO_2	0.18	0.37	1.26	......	2.17	......	0.61
H_2O-	......	1.10	1.55	0.61	....	0.84	1.16
H_2O+	12.95	14.27	13.27	12.18	13.63	11.80	12.81
Total	99.92	100.07	100.47	99.82	100.22	99.75	100.67

Kaolinite
1. Zettlitz, Czechoslovakia
2. Mexia, Texas
3. Macon, Georgia
4. St. Austell, England
5. Anna, Illinois

Anauxite
6. Bilin, Czechoslovakia
7. Ione, California

Analyses 1, 2, 6, and 7 from C. S. Ross and P. F. Kerr, *U.S. Geol. Survey Profess. Paper* 165E (1931); 3 and 4 from P. F. Kerr *et al.*, Rept. 7, American Petroleum Institute Project 49 (1950); 5 from R. E. Grim, *Econ. Geol.*, **29,** 659–670 (1934).

Table 4

CHEMICAL ANALYSES OF MONTMORILLONITE MINERALS [8]

	1	2	3	4	5	6	7
SiO_2	52.09	50.30	50.20	51.14	55.44	57.55	49.91
Al_2O_3	18.98	15.96	16.19	19.76	20.14	19.93	17.20
Fe_2O_3	0.06	0.86	4.13	0.83	3.67	6.35	2.17
FeO	....	....	....	......	0.30	0.95	0.26
MgO	3.80	6.53	4.12	3.22	2.49	3.92	3.45
CaO	3.28	1.24	2.18	1.62	0.50	1.94	2.31
K_2O	....	0.45	0.16	0.11	0.60	0.59	0.28
Na_2O	....	1.19	0.17	0.04	2.75	0.33	0.14
TiO_2	....	....	0.20	......	0.10	0.32	0.24
H_2O-	14.75	23.61	15.58	14.81	14.70	....	15.77
H_2O+	7.46		7.57	7.99		8.53	7.70
Total...	100.42	100.14	100.50	99.52	100.69	100.41	99.43

Montmorillonite
1. Tatatilla, Mexico
2. Otay, California
3. Polkville, Mississippi
4. Montmorillon, France
5. Upton, Wyoming
6. Pontotoc, Mississippi
7. Chambers, Arizona

Analyses 1 to 5 from C. S. Ross and S. B. Hendricks, *U.S. Geol. Survey Profess. Paper* 205B (1945); 6 from R. E. Grim and R. A. Rowland, *Am. Mineral.*, **27,** 746–761 (1941); 7 from P. F. Kerr *et al.*, Rept. 7, American Petroleum Institute Project 49 (1950).

CHAPTER IV

EMPIRICAL AND EXPERIMENTAL METHODS OF PREDICTING EXPANSION

Many attempts have been made to predict swelling characteristics of expansive soils based on simple index parameters. All have been able to predict, with varying degrees of accuracy, such characteristics. Yet, no one method has proven totally satisfactory, and no one method is a standard. Some of these methods are reviewed below and are followed by an explanation of the analysis method to be used in this paper.

A. Free Swell Index

The free swell test is one method of measuring the potential swelling of a soil. A known volume of dry soil is poured into a graduate which is filled with water. The loose soil is given sufficient time to settle to the bottom of the graduate and then the swelled volume of the soil is measured. The percent of free swell is calculated from the equation:

$$\text{Free Swell} = \left(\frac{V_f - V_i}{V_i}\right) 100$$

where V_i is the initial soil volume and V_f is the final soil volume. The results achieved from this test have not proven to be closely correlated to volume changes observed in more controlled expansion testing, and therefore the results of this test are very general. It has been observed that soils which exhibit free swell percentages of less than 50% seldom exhibit appreciable volume changes even under light confining pressures. Soils exhibiting free swell values as low as 100% have been known to experience considerable expansion when subjected to light confining pressures. Values of 1200% to 2000% have been observed for highly swelling soils such as bentonite [20]. The wide ranges above show that, at best, this is only an indicator type test and the test results should be used judiciously.

B. Potential Volume Change Method

Another method of identification of potentially swelling soils is the potential volume change (PVC) method developed by Lambe in work done for the Federal Housing Administration [13]. This method utilizes remolded soil samples which are first compacted in a fixed ring consolidometer under a compactive effort of 55,000 ft-lbs per cubic foot, and then subjected to a 200 psi confining pressure. Water is added and the sample is allowed to expand with the vertical expansion being monitored by a proving ring. After a 2 hour time period, the proving ring reading is taken and converted to a value of pressure. This value is designated the swell index, and utilizing Fig. 8, this value is converted to a "PVC" value. Based upon this PVC value the following classification guide can be used:

PVC Rating	Category of Expansion
2	non-critical
2-4	marginal
4-6	critical
6	very critical

Although this method has seen fairly wide usage, this classification is only a means of comparing the swelling potential of various soils and does not give a measure of the true swelling potential of any particular soil.

C. <u>United States Bureau of Reclamation Method</u>

In a paper published in 1956, Holtz and Gibbs correlated uplift pressures and volume changes occurring in samples of expansive soils to three simply determined properties. These properties are colloid content, plasticity index, and shrinkage limit. After their original attempts to correlate the results of free swell tests and volume changes (as determined by laboratory tests) produced only very general results, Holtz and Gibbs

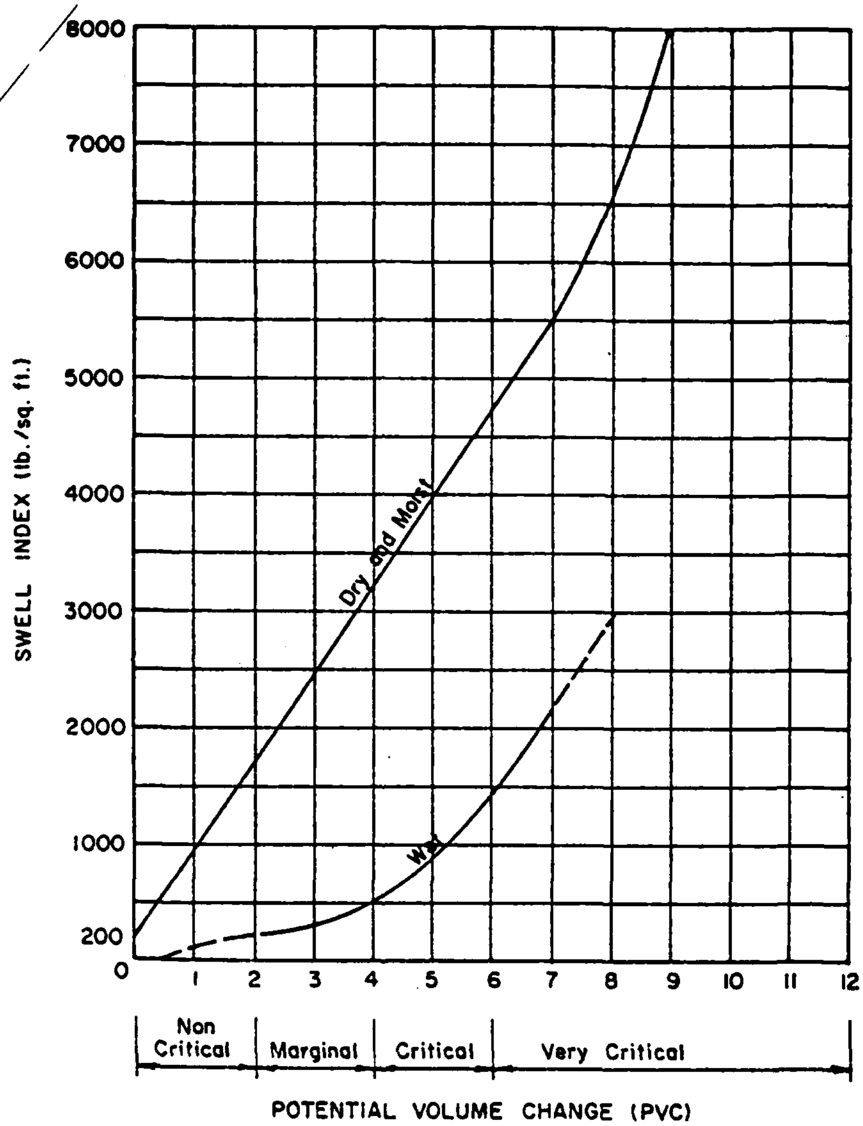


Figure 8

Swell Index versus Potential Volume Change. (From "FHA Soil PVC Meter Publication," Federal Housing Administration Publication No.701) [3] settled on these three index properties. These results show that when considered together, these index properties enable good prediction of the expansive character of soils.

The colloid content, determined from the gradation test, provides a measure of the "active" portion of the soil which is most responsible for expansive characteristics. The plasticity index provides a measure of the range of moisture change a soil can undergo and still remain in a plastic condition. Since any water either in pore water form or adsorbed form occupies a certain volume, changes in moisture content are also reflected in volume changes. Higher plasticity index values correspond to more active soils. The shrinkage limit was seen as a supplemental parameter and since it is indicative of the minimum volume to which a soil will shrink, this parameter provides a measure of the percentage of water which would be necessary to fill voids in a soil when it is at its minimum volume. With these principles in mind, tests were conducted on 38 undisturbed samples of soil. The testing was done on samples which were air dried and then allowed to become saturated under a confining load of 1 psi in a 1-D consolidometer. Figure 9 shows the test results and the classification of volume change in qualitative terms.

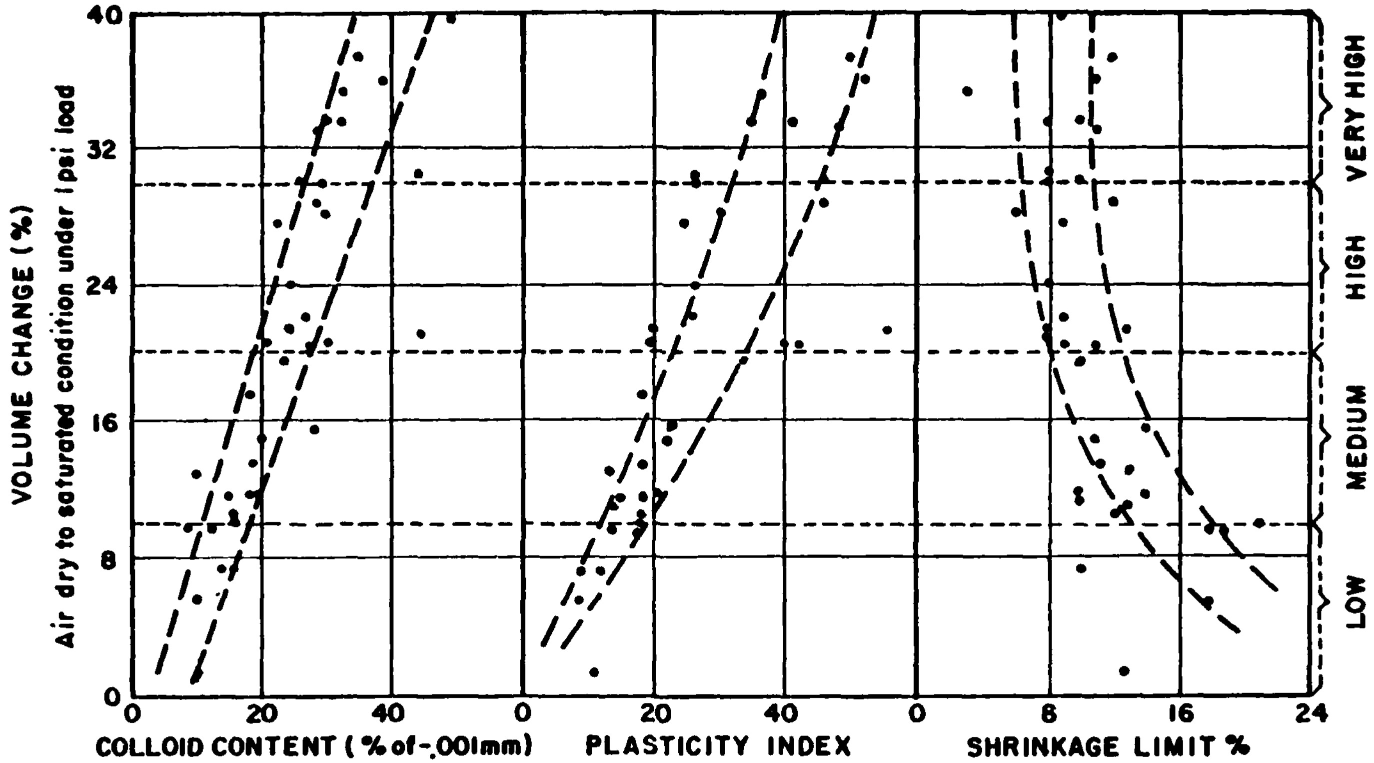


Figure 9

Relation of Volume Change to Colloid Content, Plasticity Index and Shrinkage Limit [10]

From this data, Table 5 was derived and this summarizes the classification of degree of expansion in terms of the three parameters. Holtz and Gibbs also examined other parameters as a means of predicting swelling characteristics (i.e., % of particles smaller than .005 mm, liquid limit, free swell, and montmorillonite content) but decided that the aforementioned three parameters were more advantageous for the estimation of swelling characteristics, in that they were simpler and more practical.

D. Altmeyer's Shrinkage Limit Method

Work done by Altmeyer has resulted in another, although similar, classification guide to that prepared by Holtz and Gibbs. Altmeyer sought to correlate swelling to easily determined parameters. Like Holtz and Gibbs, he too considered X-ray diffraction, microscopic examination and differential thermal analysis as too costly, in time and capital, to be of any practical value to practicing engineers. His analysis, thus, is based on more routine and simple tests.

The shrinkage limit parameter was utilized by Altmeyer but within tighter ranges than by Holtz and Gibbs. Whereas the latter proposed shrinkage limit percentages of less than 10% to be associated with a very high degree of expansion and 13% to show low degrees of

Table 5

CRITERIA FOR ESTIMATING PROBABLE VOLUME CHANGES [10]

Data from index tests			Probable expansion (percent total volume change, dry to saturated condition)	Degree of expansion
Colloid content (percent minus 0.0001 mm)	Plasticity index (percent)	Shrinkage limit (percent)		
> 28	> 35	< 11	> 30	Very high
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
< 15	< 18	> 15	< 10	Low

expansion, Altmeyer proposed the following:

Shrinkage Limit (%)	Volume Change
< 10	critical
10-12	marginal
> 12	noncritical

Altmeyer does not utilize the colloid percentage in his classification scheme, in that the hydrometer analysis test required to determine this parameter is required in order to classify soils under the Unified Soil Classification system. To determine this parameter, more than "routine" laboratory testing would be required by engineering firms, and based upon the nondistinct discrimination which these values provide (as presented by Holtz and Gibbs) they would be of questionable worth in any event. Altmeyer further contends that knowledge of colloid content alone is insufficient, and a concurrent determination of the minerals that such a colloid content represents (e.g., kaolinite, illite, or montmorillonite) is vital if this parameter is to be useful in predicting or classifying swell characteristics. Hence, he proposes using the parameter linear shrinkage, which is the percentage of linear shrinkage which a soil mass experiences when it is reduced from some upper moisture value (generally the field moisture content) to the shrinkage limit. The shrinkage limit is the lower moisture content limit below which no volume change occurs in the soil sample. From testing on large numbers of samples in the greater Los Angeles area, Altmeyer proposes the following classification scheme:

Linear Shrinkage (%)	Volume Change
> 8	critical
5 - 8	marginal
< 5	noncritical

The last classification scheme offered by Altmeyer is based upon volume changes observed under testing of specimens in fixed ring consolidometers under loads of 650 PSF. This value of normal load was selected as representative of dead loads on footings of wooden framed single story structures. Considering that the expansion realized is highly dependent on moisture content and densities (Figure 10) several representative combinations of these parameters were examined and the following scheme was proposed.

% Expansion[3]	Volume Change
> 1.5	critical
.5 - 1.5	marginal
< .5	noncritical

[3]These values are substantially less than those presented by Holtz and Gibbs but are attributable to the higher confining pressure of 650 PSF vs 144 PSF used by Holtz and Gibbs.

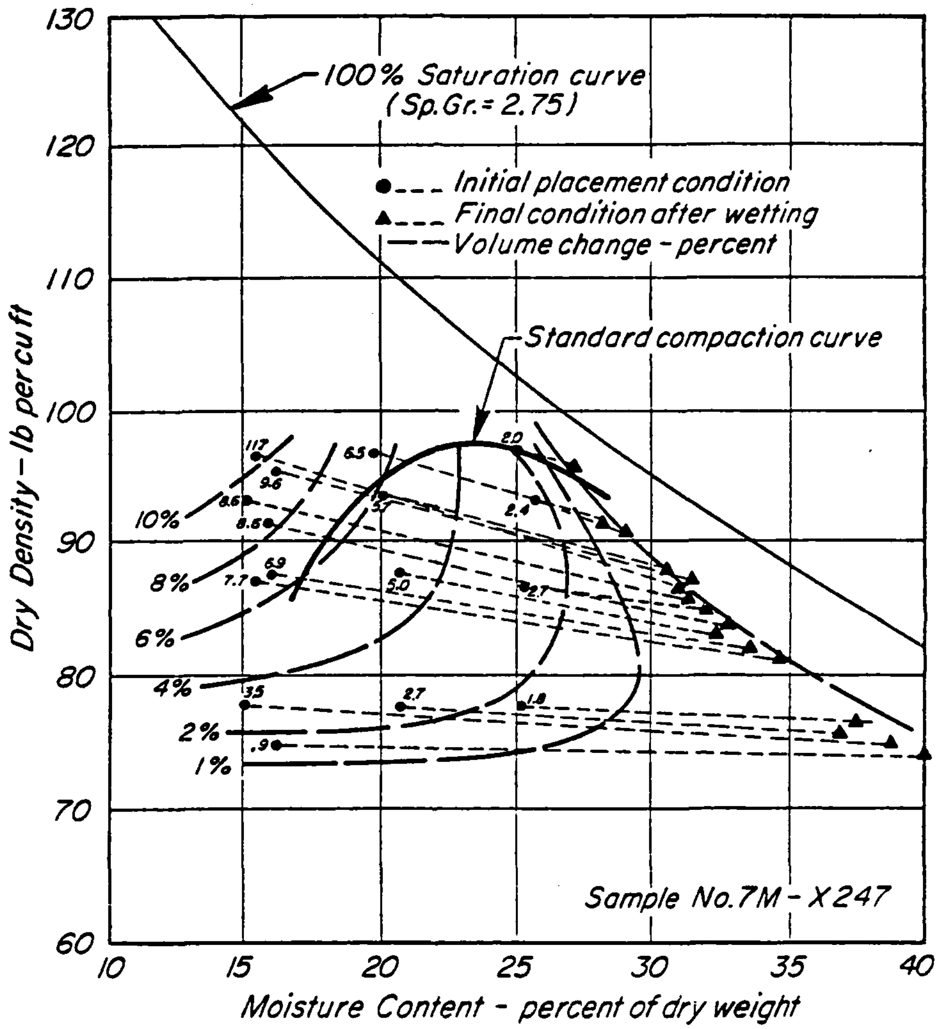


Figure 10

Percentage of Expansion for Various
Placement Conditions when Under
1-lb. per sq. in. Load [10]

E. Seed, Woodward and Lundgren "Activity" Method

The approach of Holtz and Gibbs provided a valuable means of predicting swelling characteristics. However, as pointed out by Seed, Woodward and Lundgren (Table 6) prediction of swelling using this method can result in conflicting conclusions. It was such inconclusiveness in prediction which prompted these individuals to seek a more reliable method of predicting expansive characteristics.

To begin their analysis, a differentiation between the "swell" and the "swelling potential" of a soil was made. Using the conclusions found by Holtz and Gibbs that the swell of a soil is influenced not only by soil classification indices, but by such factors as emplacement condition (dry density and water content - see Fig. 10), the method of placement and environmental conditions (i.e., the availability of moisture), Seed, Woodward and Lundgren divided the expansion characteristics of any soil into two distinct parts. The first is what they termed swelling potential of the soil, which is a measure of the ability or the capacity of the soil constituents to promote swelling. The second is the degree to which this capacity is realized in actual soils in the field, as dictated by the environmental and placement conditions. They limited themselves to examining the former of these factors, and

Table 6

PREDICTION OF EXPANSION CHARACTERISTICS USING BUREAU OF RECLAMATION CORRELATIONS [22]

Sample No.	Liquid Limit	Plasticity Index	Shrinkage Limit	Percent Finer Than 0.001 mm	Potential Expansibility		
					Based on Plasticity Index	Based on Shrinkage Index	Based on Percent Finer Than 0.001 mm
645-3-1-3	41	19	19.2	28	Medium	Low	High
640-1-1-3	55	34	15.3	39	High	Low	Very High
649-13-1-3	32	14	14.2	26	Low	Medium	High
619-5-1-3	30	12	6.3	12	Low	Very High	Low
8-1-1-3	39	12	13.4	29	Low	Medium	High
11-1-1-3	41	19	17.5	26	Medium	Low	High
15-2-1-3	19	3	14.8	26	Low	Medium-Low	High

further conducted testing on laboratory prepared samples in order to avoid soil variability which would have been included had they used only natural soils. Seven different clay "soils" were prepared by combining, in varying proportions, the three basic clay minerals, kaolinite, illite and montmorillonite (the latter in the form of Wyoming betonite). Swell potential was then defined as the percent swell of a laterally confined sample on soaking under a 1 psi surcharge after being compacted to maximum density at optimum water content in the standard AASHO compaction test. In addition to the swell potential, liquid limit, plastic limit, shrinkage limit and grain size distribution were determined for each soil.

The laboratory soils were prepared and for each soil type, the amount of swell exhibited by each increased as the percentage of clay increased. These increases were found to be such that on a log-log plot of swell percent versus percent clay size (<.002 mm) each soil plotted as a straight line (Fig. 11). The equation describing these lines takes the general form:

$$S = KC^{x} \text{ or } \log S = \operatorname{Log} K + x \log C$$

where S = swelling potential measured as the percent swell under 1 psi for samples prepared at optimum moisture content and maximum density in the Standard AASHO compaction test. C = percent clay sizes

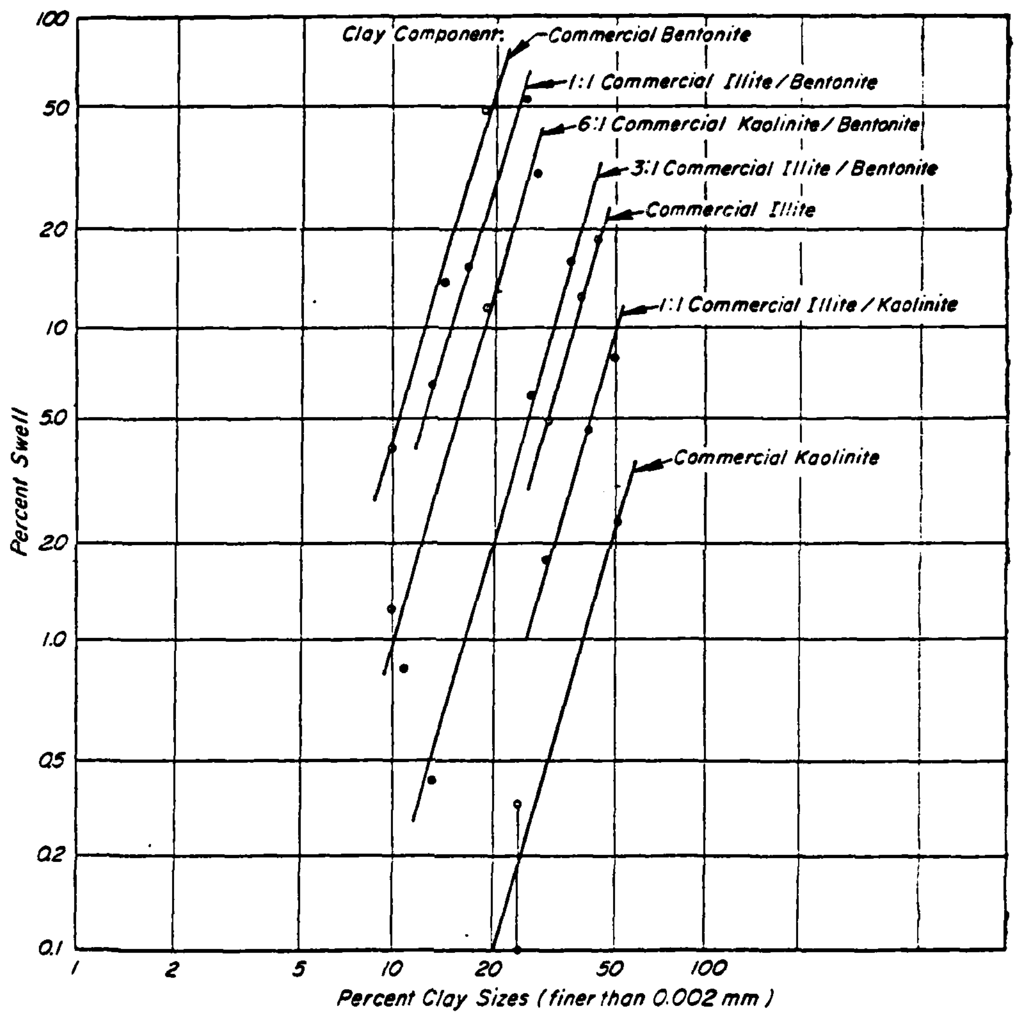


Figure 11

Relationship Between Percent Swell and Percent Clay Sizes for Experimental Soils [22]

(< .002 mm); x = slope of the line; and K = coefficient evaluated by the value of S when $C = 1\%$, (i.e., the S axis intercept).

Interestingly enough all of the samples produced lines which when plotted were parallel to each other. The exponent X was determined to be very nearly 3.44 for each soil and assumed to be a constant for any type clay. Thus the coefficient K was determined to be the only factor differentiating any two clays. Evaluating this K for these test soils indicated that as the swell potential increased so did the numerical value of K.

In order to establish a simpler means of evaluating the K for each soil, a relationship between K and the activity of the soil was explored. The activity of a soil is a parameter which was introduced in 1953 by A.W. Skempton and defined as the ratio of the plasticity index to the clay size fraction of a soil. Thus plots of plasticity index versus percent clay sizes were made for these soils as shown in Figure 12. It can readily be seen that these lines did not originate at the origin of this plot (Skempton had extrapolated his data points and found them to originate at the origin. See reference [24].) And so the definition of "activity" was refined to be the ratio of the change in plasticity index to the change in clay content $\frac{\Delta PI}{\Delta C}$. It is thus a more general

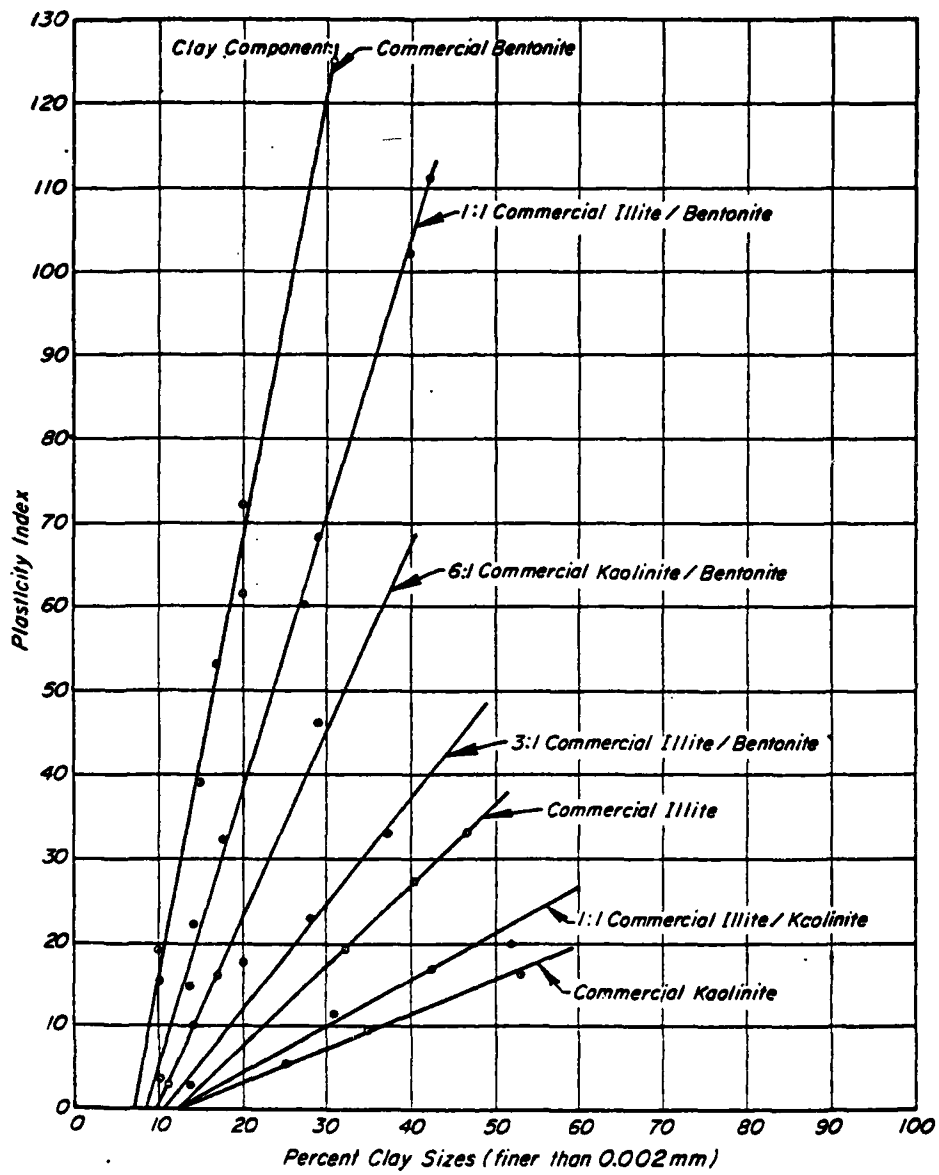


Figure 12

Relationship Between Plasticity Index and Percent Clay Sizes for Experimental Soils [22] definition in that the line relating plasticity index to percent clay content does not have to pass through the origin. Figures 12 and 13 indicate that there indeed should be a relation between activity and the value K, as each clay retained the same relative position on each plot. Furthermore, plotting the value of K from Figure 11 versus activity (slope of line in Figure 12) on log-log scale produced a straight line relationship verifying a relationship (Fig. 14). From this plot it was found that:

$$K = CA^{y} = 3.6 \times 10^{-5} A^{2.44}.$$

Substituting this value into the earlier equation resulted in

$$S = (3.6 \times 10^{-5})(A^{2.44})(C^{3.44}) \qquad \text{(eqn.1)}$$

as an expression of the swelling potential of any soil. Further, a family of curves on an activity versus percentage clay sizes plot, which separates swelling potential qualitatively (i.e. low, medium, high, and very high) was developed for practical engineering use (Fig. 15).

Since these results had been obtained using artifically prepared soils, the results were applied to a series of natural soils in order to verify their applicability. In order to determine the "activity" of

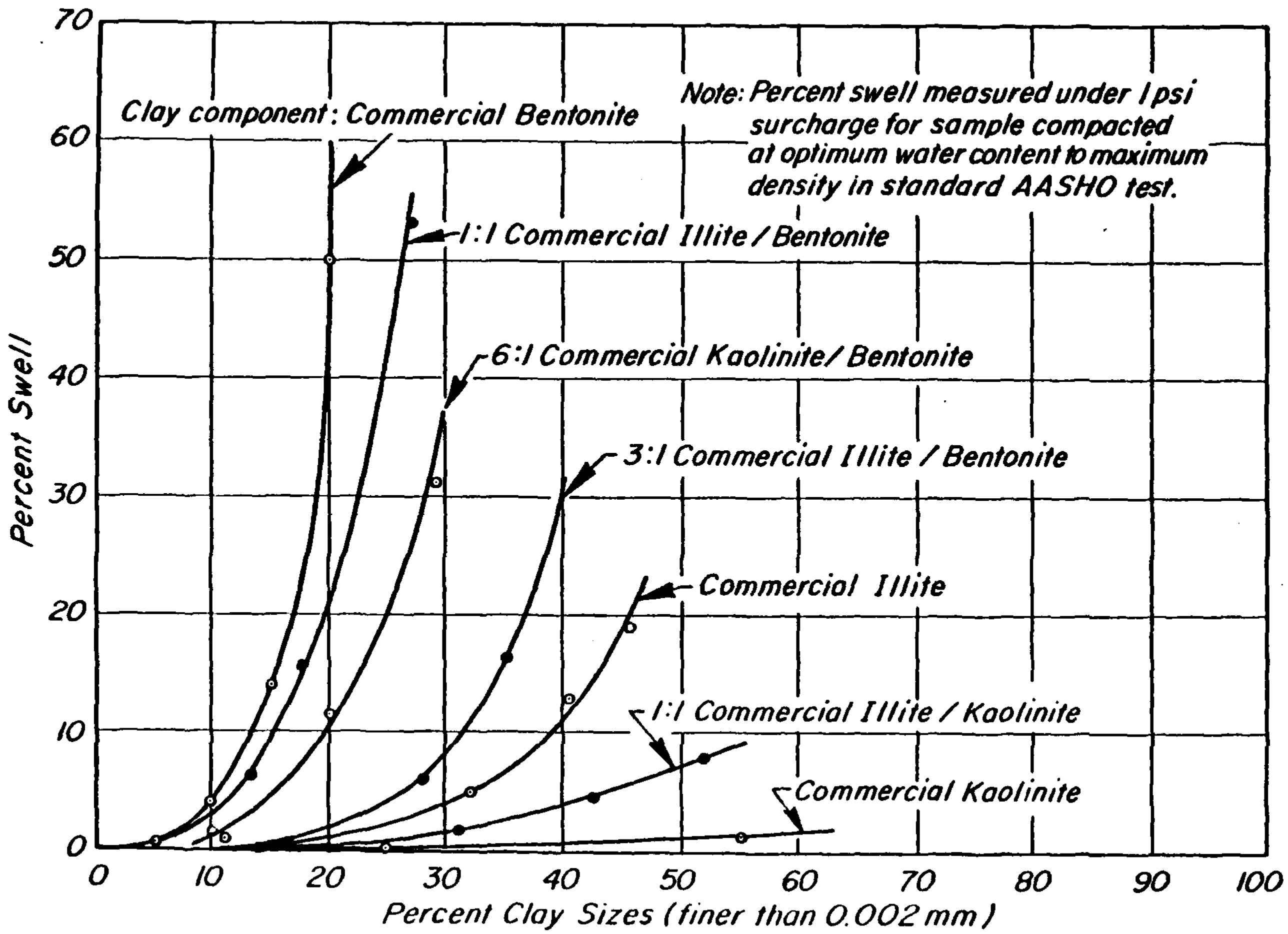


Figure 13

Relationship Between Percent Swell and Percent Clay Sizes for Experimental Soils [22]

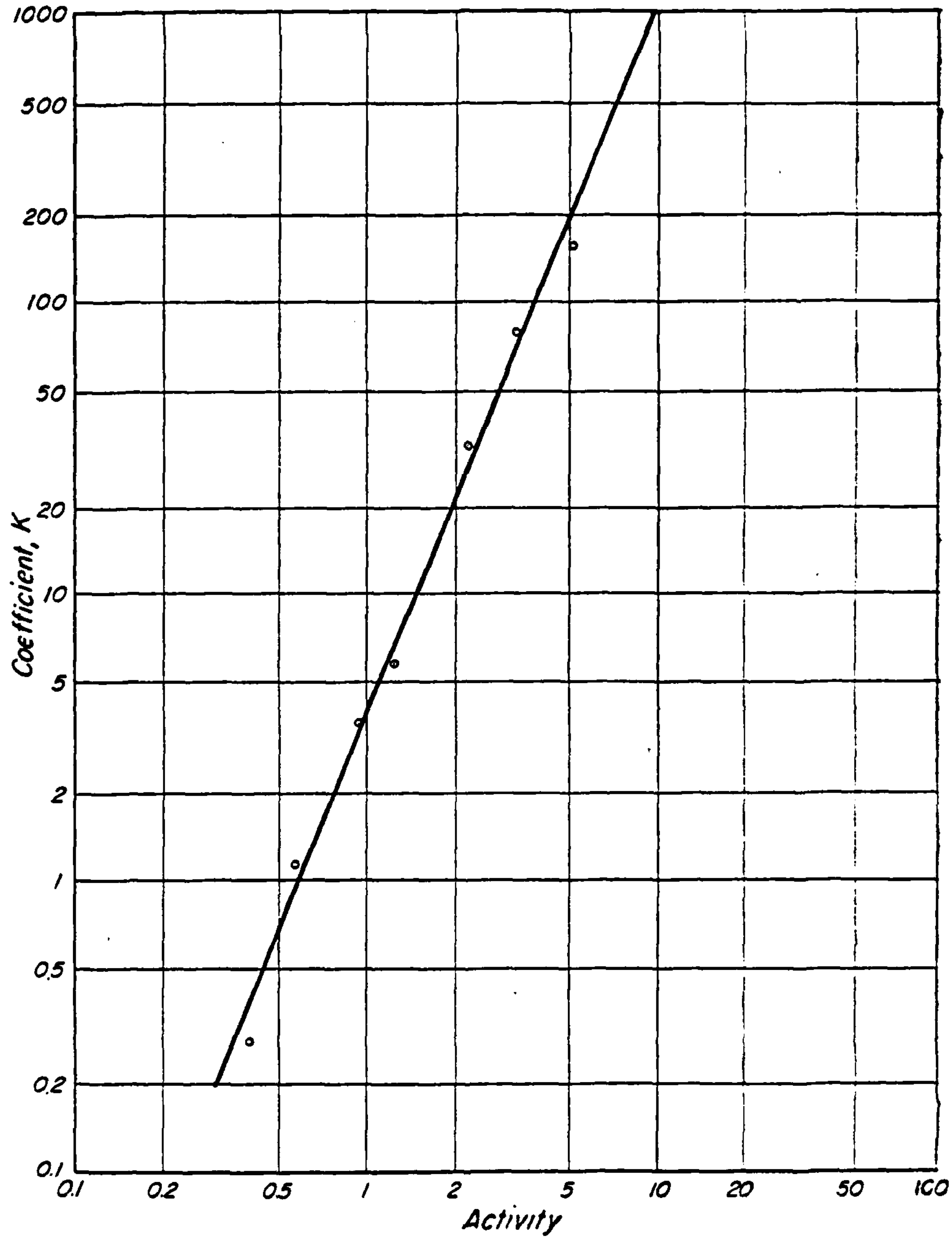


Figure 14

Relationship Between Coefficient K and Activity for Experimental Soils [22]

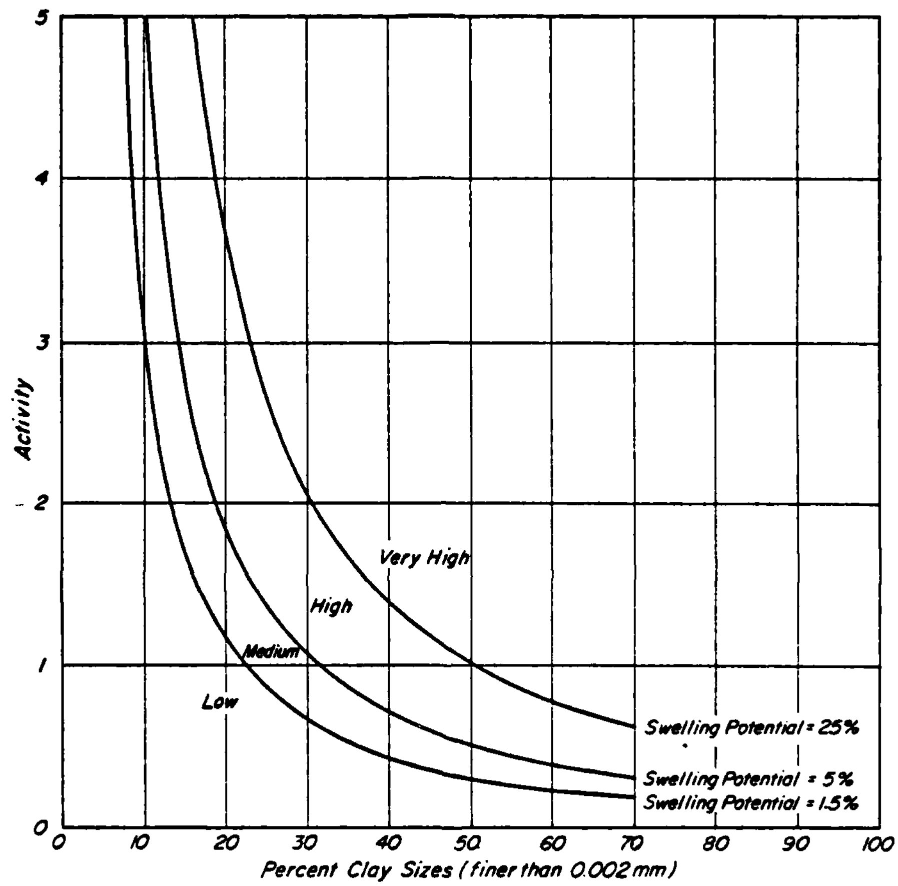


Figure 15

Classification Chart for
Swelling Potential [22]

a soil as defined above $(\frac{\Delta PI}{\Delta C})$ and not have to undertake the testing required to generate a curve like Figure 14, the authors proposed re-defining activity as PI/C-10 for artifically prepared soils and PI/C-5 for natural soils. (See reference [22] for more detailed explanation.) Using this relationship, the authors found excellent correlation between their family of curves for qualitative prediction and some 27 natural soils (Fig. 16). They thus propose Fig. 15 as a guide for the prediction of swelling potential.

F. Seed, Woodward & Lundgren "Plasticity" Method

From the soil test results obtained above, a trend was seen to exist between swell potential and plasticity index (Fig. 17). In light of this, predictive equations were sought relating these two quantities. Utilizing the redefinition of $A = \frac{PI}{C-n}$ and inserting this into the equation $S = KA^{2.44} C^{3.44}$ results in the expression $S = K(PI^{2.44})N$ where $N = C^{3.44}/(C-n)^{2.44}$. When the value of N is plotted versus percent clay sizes for various n values, the curves on Fig. 18 are obtained. For $n = 10$ (which was the average value assumed for artifical soils used in the research), N is a relative constant value for percent clay sizes between 19% and 70%, varying between 80 and 120. Thus, using $N_{average} = 100$ for soils with clay contents in this range, swell potential

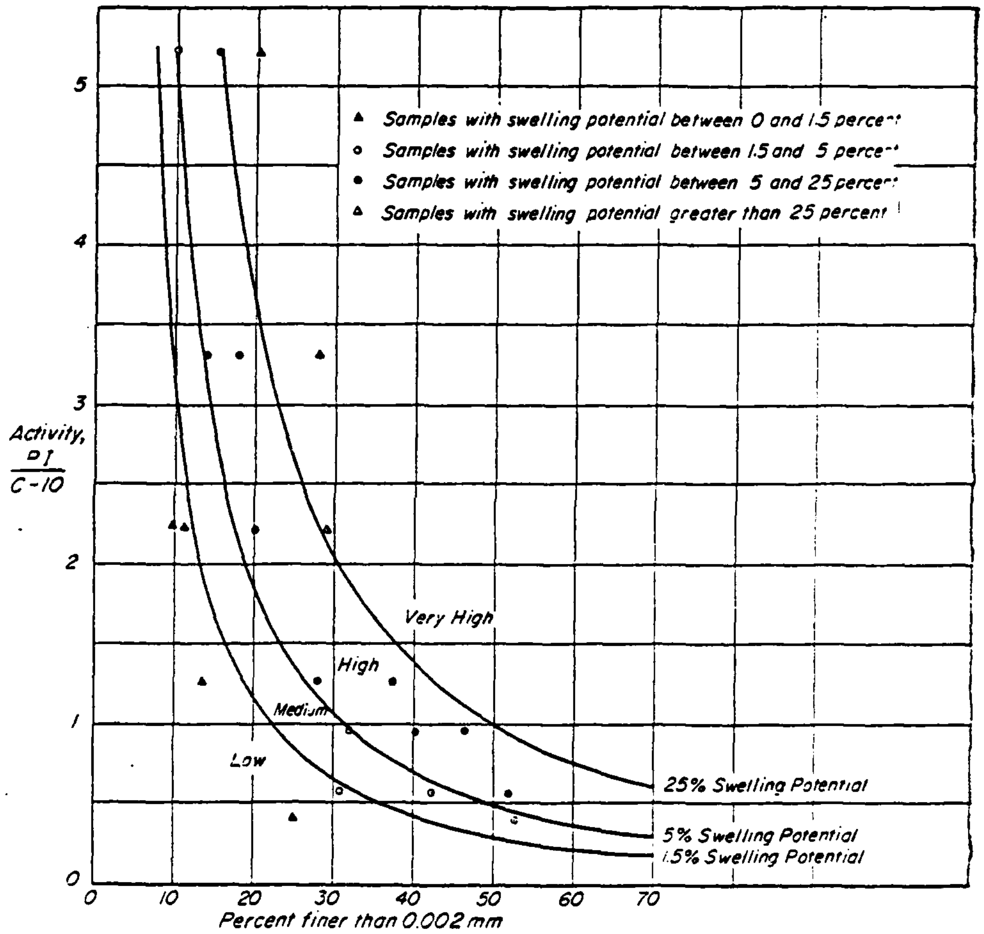


Figure 16

Applicability of Chart for Classification of 21 Experimental Soils [22]

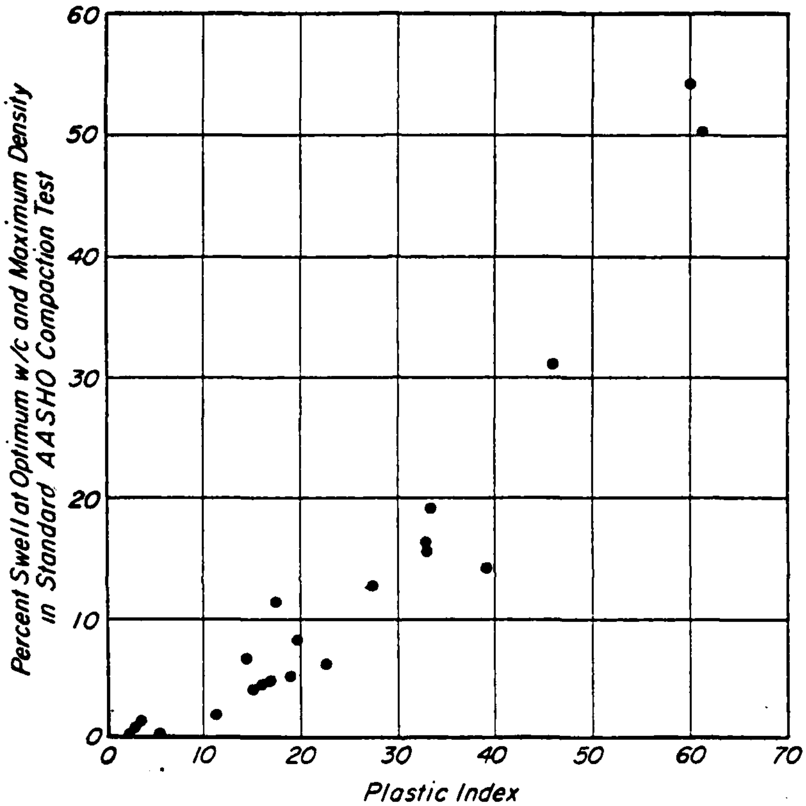


Figure 17

Relation of Swell Potential to Plasticity Index [22]

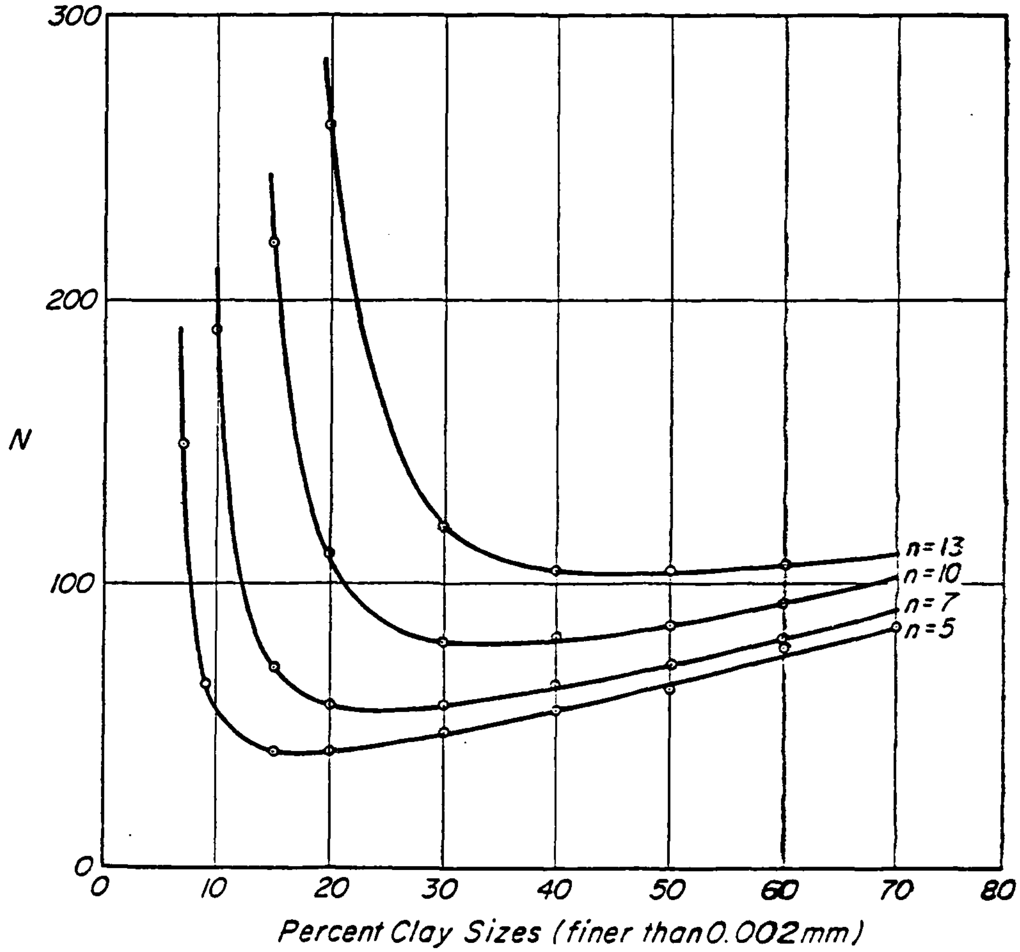


Figure 18

Relationship Between Parameter N
and Percent Clay Sizes [22]

may be obtained using the formula $S = 100K(PI)^{2.44}$. Likewise, using $N = 5$ (which the authors proposed as an average value for natural soils), $N_{average} = 60$ for soils in the clay content range of 8% to 65%. Consequently $S = 60\,K(PI)^{2.44}$ is a good approximation for natural soils in this range. Agreement of these equations were found to be $\pm$ 20% and $\pm$ 33% respectively when compared to values obtained utilizing formula (1).

G. Vijayvergiya and Ghazzaly Method

Another effort to predict swelling potential of clays was made by Vijayvergiya and Ghazzaly in which they examined clay samples with the intent of correlating the swelling potential to routine physical properties or classification categories. The difference in this and earlier attempts is that their test samples were all natural clays as opposed to prepared clay samples used for testing in other efforts.

Having reviewed other research attempts made to predict the behavior of natural expansive clays, the authors selected several easily determined soil index properties for further examination. Since previous research had shown the plasticity index to be of value in examining swelling behavior (qualitatively reflecting the amount and type of clay mineral present in the soil) and also since there is a linear relation between plasticity index and liquid limit, liquid limit was selected as one of the parameters to be examined. Since highly dessicated clay soils which have great expansive characters are generally quite dense and have comparatively low moisture contents, these two parameters were also felt to be worthy of examination.

The test data was comprised of soil testing results from published works as well as results from the case files of McClelland Engineers, Inc. of Houston, Texas. Geographically nearly 89% of the 270 samples were Texas soils and the remainder represented Israel, California, Oklahoma and Arkansas. Additionally, the undisturbed samples selected were from depths of less than 10 feet so as to minimize the effects of stress relief. The soils all fell above the "A" line of the Unified Soil Classification System. The soils were separated in terms of liquid limits into groupings having a range of $\pm$ 2 (i.e. for analysis purpose LL = 50 covers the range 48 to 52). Analysis was done and correlations developed between percent swell and water content, percent swell and dry unit weight, swell pressure and water content, and swell pressure and dry unit weight. In each case, the liquid limit values served as an additional discriminator. Once data points were plotted, straight lines were fitted and a family of curves was obtained, from which predictive equations were extrapolated (Figures

19 and 20). The Figures show a nice correlation and the authors reported correlations factors of 0.7 or better for these lines. They conclude from their analysis that the above mentioned parameters can be used to predict either swelling pressure or percent swell, although they place more confidence in the correlations using water content than those using dry density, as they feel the former can be more reliably determined.

H. Chen's Analysis

In his text on expansive soil Chen recognizes, as have others, the problems caused by expansive soils and that no definite method of measuring the swelling potential has been established. The correlation of results from each effort has been hampered by different definitions used to express swelling potential. Additionally, there is a multitude of mineralogical factors which affect the swelling potential of a natural soil, and this large number of variables further clouds the issues.

Environmental factors such as surcharge pressure, direction and degree of saturation, initial moisture content, soil strata thickness, in situ dry density, and time also have an influence on the amount of swell which is realized. Consequently, Chen concludes that any attempt to predict swelling potential in light of this plethora of variables will be extremely difficult,

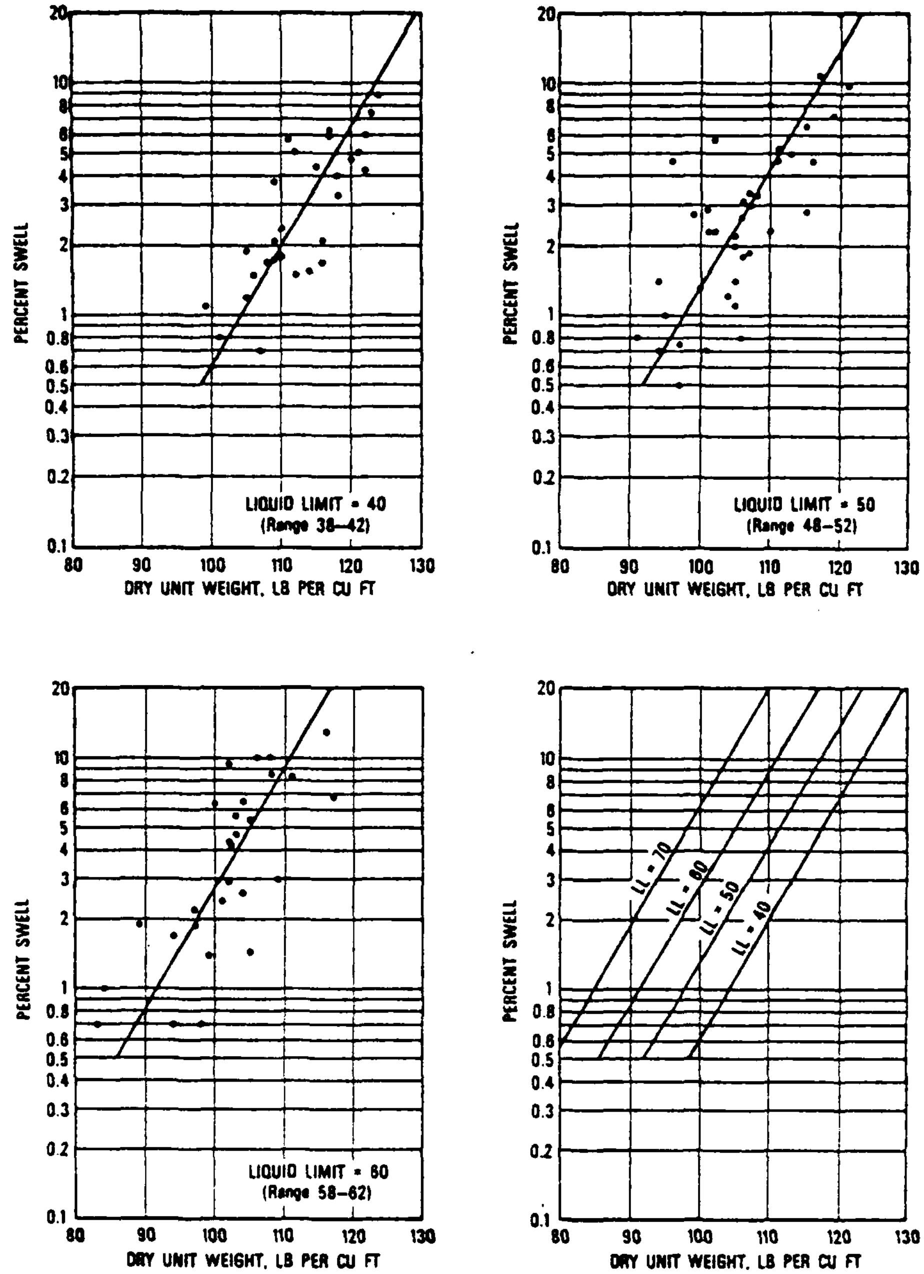


Figure 19

Correlation of Percent Swell with
Liquid Limit and Dry Unit Weight [27]

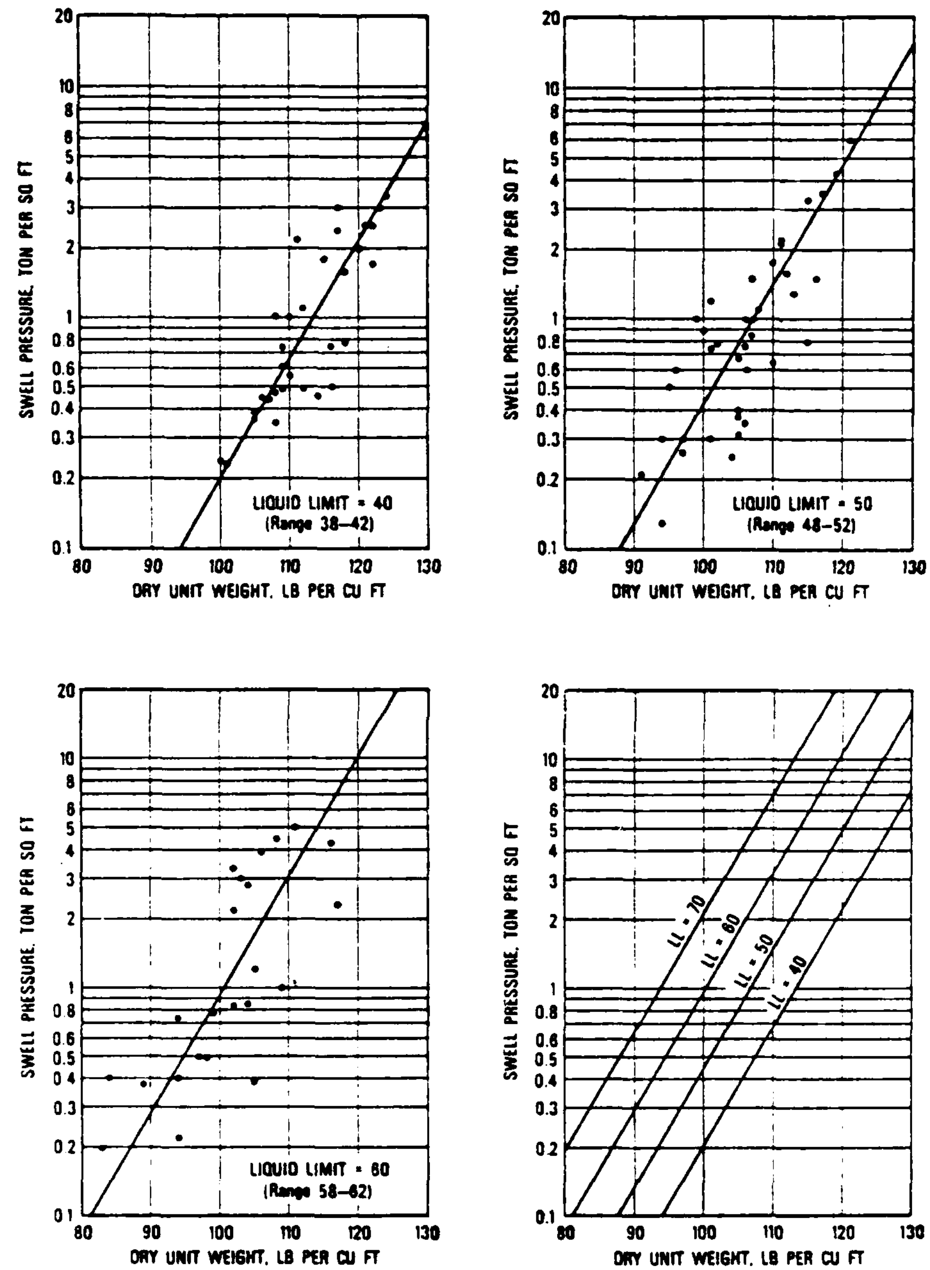


Figure 20

Correlation of Swell Pressure with
Liquid Limit and Dry Unit Weight [27]

if not unreliable. He therefore seeks to prove that another parameter, namely swell pressure, will be more reliable in predicting swelling behavior than will swell potential. To do this he conducts a series of tests on a specific expansive soil sample found in Denver, Colorado, varying only one variable at a time and measuring swelling performance. For the following tests, the swelling pressure remained a constant, within experimental error:

a. varying the percent saturation while keeping initial density and moisture content constant (Figure 21 and Table 7)
b. varying moisture content while maintaining initial density constant (Figure 22 and Table 8)
c. varying sample thickness while keeping initial density and moisture contents constant (Figure 23 and Table 9).

However, when initial density was increased for constant initial moisture content, a marked increase in the swelling pressure was also noted (Figure 24 and Table 10). This series of tests allowed Chen to conclude that the swelling pressure is essentially a constant for a given soil and varies only as the dry density changes (Fig. 25). Hence, for undisturbed soils the in situ dry density can be used to quantify the swelling pressure and thus the

Table 7

EFFECT OF VARYING DEGREE OF SATURATION ON VOLUME CHANGE AND SWELLING PRESSURE FOR CONSTANT DENSITY AND MOISTURE CONTENT SAMPLES [3]

Moisture content, percent		Initial density, pcf	Volume increase, percent	Swelling pressure, psf	Degree of saturation, percent
Initial	Final				
9.66	13.07	106.6	1.83	16,000	61.0
9.66	14.53	106.0	3.35	15,500	67.0
9.66	17.58	105.6	4.35	12,000	82.0
9.66	18.50	106.7	5.53	17,000	86.3
9.66	19.93	105.9	6.25	15,000	93.0
Average 9.66		106.2		15,100	

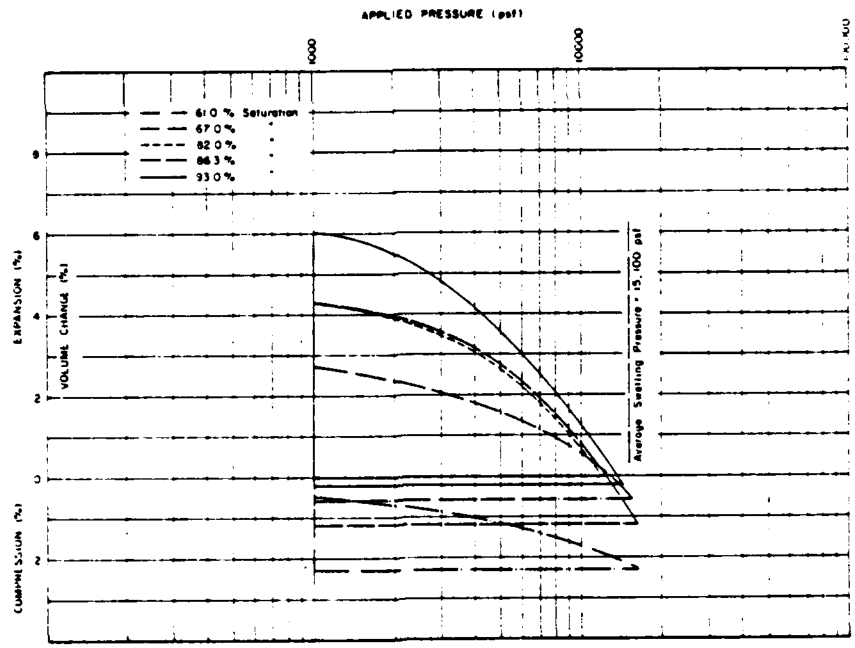


Figure 21

Relationship Between Degree of Saturation and Volume Increase for Constant Density and Moisture Content Samples [3]

Table 8

EFFECT OF VARYING MOISTURE CONTENT ON VOLUME CHANGE AND SWELLING PRESSURE FOR CONSTANT DENSITY SAMPLES [3]

Initial density, pcf	Moisture content, percent		Volume increase, percent	Swelling pressure, psf
	Initial	Final		
106.97	5.84	20.34	7.71	9,500
105.93	9.95	20.77	5.55	9,500
106.27	10.77	18.75	5.03	12,500
105.60	12.48	22.09	4.30	9,500
106.47	12.92	20.54	3.48	9,000
106.37	14.84	19.59	3.30	10,500
105.46	17.97	18.50	2.15	7,000
105.73	18.59	19.41	1.38	7,500
106.35	19.37	20.18	0.75	9,000
Average 106.13		20.02		9,333

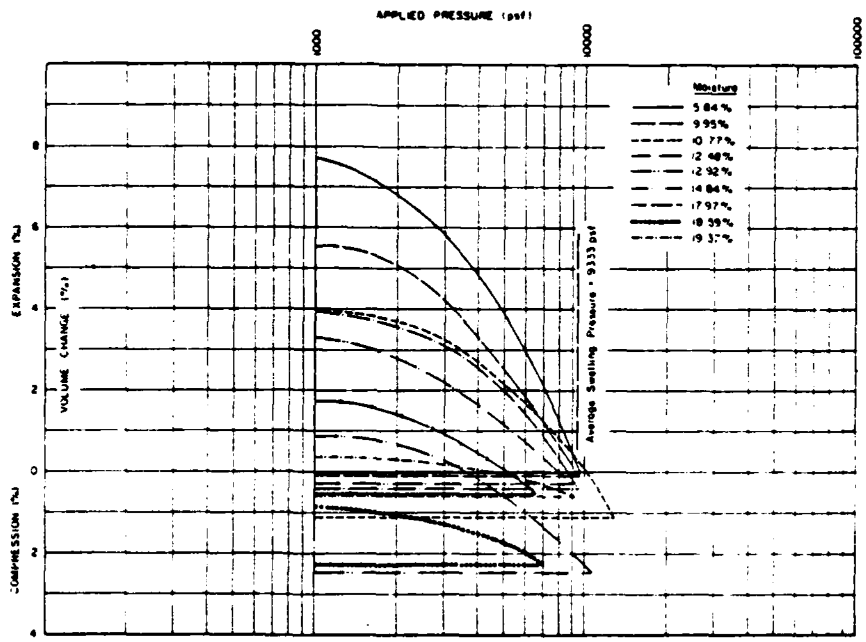


Figure 22

Relationship between Initial Moisture Content and Volume Increase for Constant Density Samples [3]

Table 9

EFFECT OF VARYING SAMPLE THICKNESS ON VOLUME CHANGE AND SWELLING PRESSURE FOR CONSTANT DENSITY AND MOISTURE CONTENT SAMPLE [3]

Initial density, pcf	Moisture content, percent		Sample thickness, in.	Volume increase, percent	Volume increase, in.	Swelling pressure, psf
	Initial	Final				
105.20	10.10	22.30	0.504	5.66	0.0285	11.000
106.33	10.10	20.92	0.748	5.75	0.0430	11.500
105.31	10.10	21.14	1.007	5.15	0.0520	11 [illegible]
106.05	10.10	20.49	1.250	5.60	0.0700	15.000
106.05	10.10	20.58	1.500	5.60	0.0840	12.500
Avg. 105.78	10.10	21.08		5.54		12.200

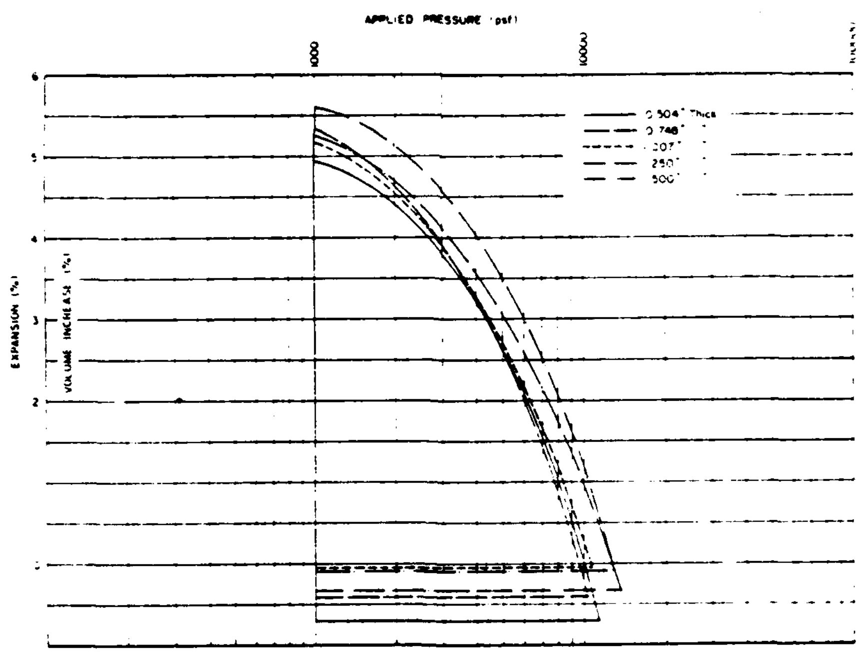


Figure 23

Relationship Between Sample Thickness and Volume Increase for Constant Density and Moisture Content Samples [3]

Table 10

EFFECT OF VARYING DENSITY ON VOLUME CHANGE AND SWELLING PRESSURE FOR CONSTANT MOISTURE CONTENT SAMPLES [3]

Initial density, pcf	Moisture content, percent		Initial degree of saturation, percent	Volume increase, percent	Swelling pressure, psf
	Initial	Final			
94.3	12.93	21.27	45.0	2.7	2,600
99.4	12.20	24.92	48.1	3.8	4,600
100.2	12.93	19.93	52.1	4.2	5,000
103.3	12.93	20.51	56.3	5.1	7,000
109.1	12.93	20.56	65.4	6.7	13,000
110.8	12.20	19.03	64.7	7.3	14,000
114.5	12.20	19.17	71.6	8.2	21,000
118.9	12.20	17.08	81.2	8.6	35,000
Average	12.55	21.08			

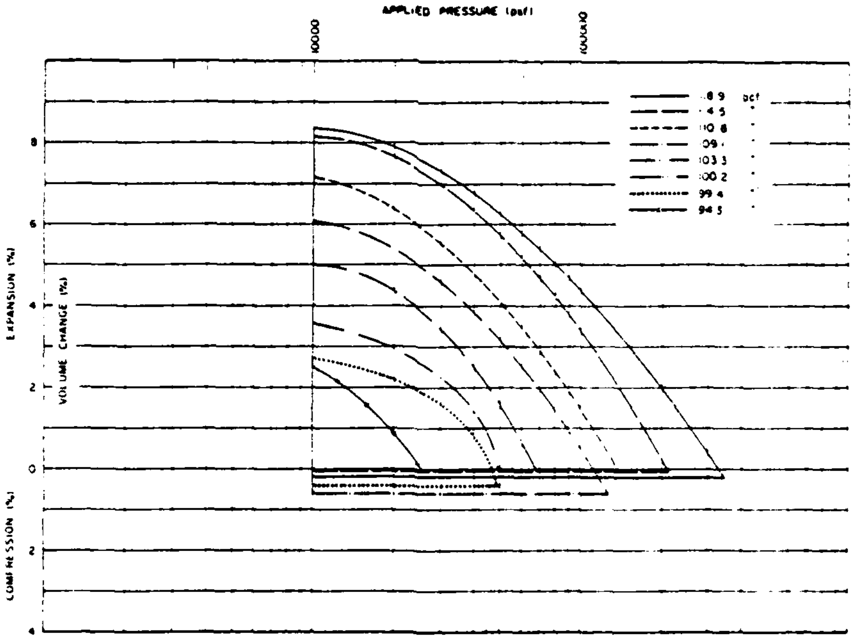


Figure 24

Relationship Between Density and Volume Increase for Constant Initial Moisture Content Samples [3]

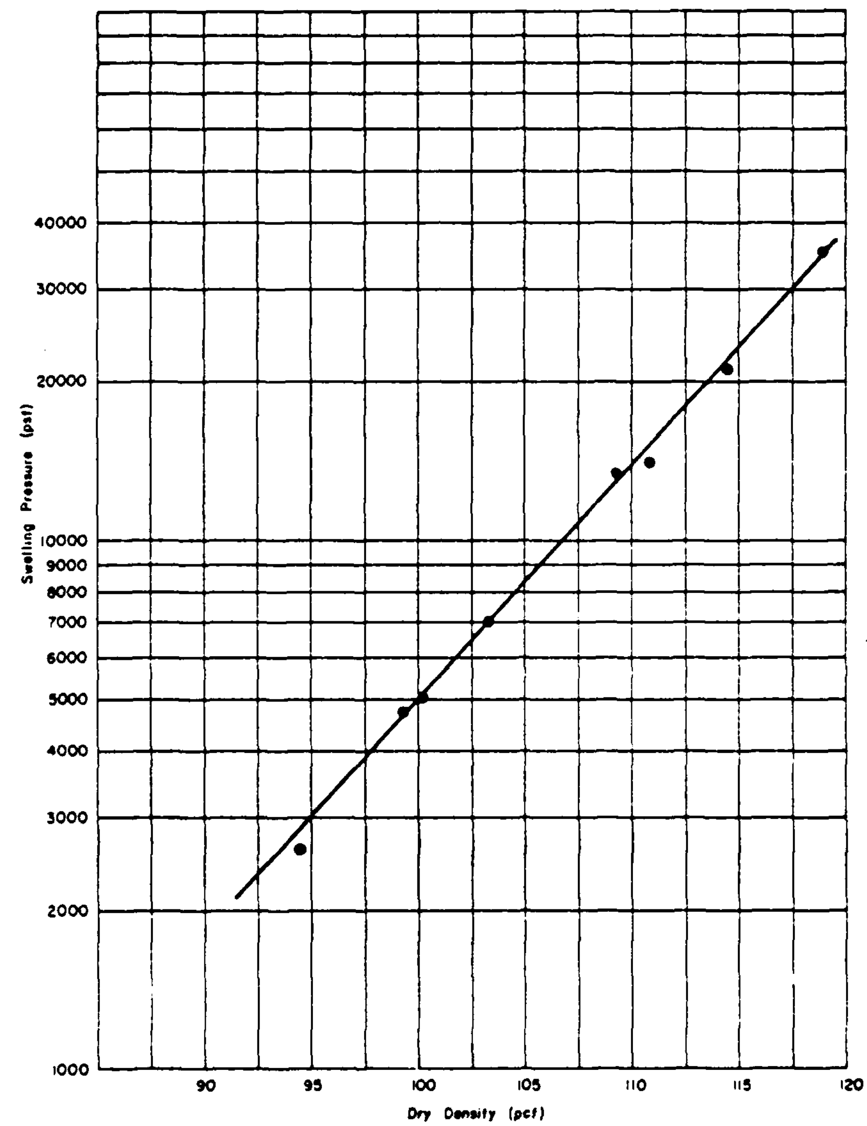


Figure 25

Effect of Varying Density on Swelling Pressure for Constant Moisture Content Samples [3] swell characteristics of a given soil. Among other conclusions, Chen felt that although only one soil was so tested, all expansive soils will behave in similar fashion, and that swelling pressure is the basic physical property of expansive soils.

CHAPTER V

DATA ANALYSIS AND CONCLUSIONS

A. Purpose and Analysis Method Selection

The purpose of this report is to provide a means of predicting the swelling characteristics of soils found in the Rocky Mountain area. In order to accomplish this, a base of data was required for analysis. Mr. Fu Hua Chen of Chen and Associates, Inc., a consulting soils engineering firm in Denver, Colorado, provided access to the records of his firm. Utilizing this data, two methods of predicting swelling characteristics of these soils have been examined. The selection of these methods is explained below.

Chen's testing [3] shows swelling pressure to be a unique property of a soil which is useful in the estimation of soil swelling. Thus, this parameter is incorporated into the analysis methods used herein. Vijayvergiya and Ghazzaly [27] predict swelling pressures using dry density and liquid limit values, and their method serves as one of the methods used in this report. The second method to be used parallels the first, except that instead of using the liquid limit value as the discriminating parameter, a parameter PI/-200 is introduced. The activity parameter (PI/percentage of clay sized particles) would have been used as the results of Seed, Woodward and Lundgren [22] indicate that it, too, is useful in the prediction of soil swelling. However, the empirical data of Chen and Associates, Inc. do not include the percentage of clay sized particles, and thus, a determination of activity for each soil is not possible. Chen [3] contends that the percentage of particles passing the #200 sieve is useful in predicting expansion (Table 11). Thus the parameter, PI/-200, which relates plasticity index to the percentage of particles passing the #200 sieve, is proposed. This parameter (a "pseudo activity") is used in conjunction with dry density to predict swelling pressures in the second analysis method.

B. Data Acquisition

Appendix A summarizes the data selected from Chen's files. For convenience, the data is arranged according to increasing values of liquid limit. Four items of information are required . r any soil sample to be included among the data in Appendix A. The dry density, liquid limit, percentage of the sample which passes the #200 sieve, and the results of a swell-consolidation test are required. The combination of this data allows for the classification of the sample according to the Unified Soil Classification System (USCS) and also meets the needs of the analyses outlined above. All soil

Table 11

DATA FOR MAKING ESTIMATES OF PROBABLE VOLUME CHANGES FOR EXPANSIVE SOILS [3]

Laboratory and field data			Probable expansion, percent total volume change	Swelling pressure, ksf	Degree of expansion
Percentage passing No. 200 sieve	Liquid limit, percent	Standard penetration resistance, blows/ft			
>35	>60	>30	>10	>20	Very high
60-95	40-60	20-30	3-10	5-20	High
30-60	30-40	10-20	1-5	3-5	Medium
<30	<30	<30	< 1	1	low

samples which exhibit swelling fall above the "A" line in the USCS scheme. The files of Chen futher distinguish these soils into clays and claystones. The distinction is that claystones represent samples of sedimentary rocks which are high in montmorillonite content. Bedrock shales of the Pierre, Laramie and Denver formations are examples of this type of rock, and are common in the Rocky Mountain region. These claystones can exhibit significant degrees of swelling. Thus the data shown in Appendix A is grouped into clays and claystones.

The values of liquid limit, natural dry density and the percentage passing the #200 sieve are taken as reported in the records of Chen and Associates. The value of swelling pressure for each sample is interpolated from a plot of the swell-consolidation test, and thus involves some judgement on the part of the author. A typical swell test is shown in Figure 26. The manner of determining swelling pressure is shown on this figure, and is further explained in Section C.

C. Test Procedures and Apparatus

There is at present no universally used test for the determination of swelling characteristics of a soil, and consequently research results are not readily correlated. The testing done by Chen and Associates, Inc. is conducted at a confining pressure of 1000 psf (6.94 psi),

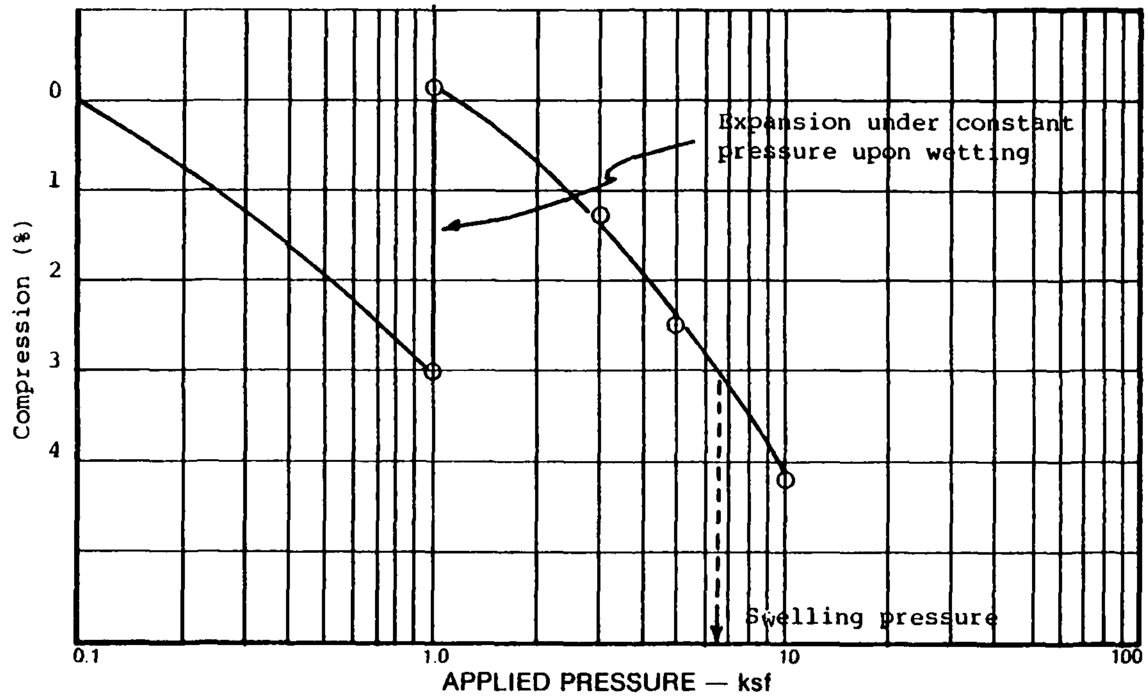


Figure 26

Determination of Swelling Pressure from Results of Typical Swell - Consolidation Test and thus all values presented herein are for this confining pressure. The test procedure used to generate the data is to place an undisturbed sample in a consolidometer under the above 1000 psf surcharge for 24 hours, and record the deformation. The sample is then allowed to become saturated while still under this surcharge load, and measurements of the expansion are recorded. Once all swelling has taken place, the load on the sample is increased, and measurements of the resulting volume changes are made. Successive increases in the surcharge load allow for the determination of the "reloading" portion of the curve shown in Figure 26. The intersection of this portion of the curve with the abscissal line (i.e., return of the sample to its initial volume prior to saturation) yields the swelling pressure of the soil. The test apparatus most often used in this process is the simplified lever-type consolidometer shown in Figure 27.

D. Data Analysis

For each of the soil categories (i.e., clays and claystones) two analyses are accomplished. The first follows the method of Vijayvergiya and Ghazzaly. Soils in each category are first separated into ranges of liquid limit (e.g. 20-29, 30-39, 40-49 etc.), and are then plotted on a semi-logarithmic scale of swelling pressure versus natural dry density. Once all data points within

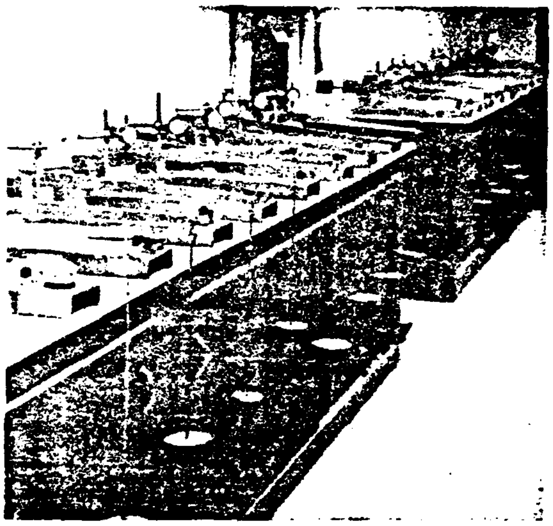


Figure 27
Simplified Lever-type Consolidometer [3]

a given range are plotted, straight lines are fitted to the data by linear regression analysis, as well as visually (see Figures 28-32 for typical results). The purpose of the dual fitting of lines is to see if a more reliable method than the visual one used by Vijayvergiya and Ghazzaly will produce the same parallelism of lines which these individuals achieved (Figs. 19 and 20). The results obtained by utilizing the linear regression process do not produce this nice parallelism (Figs. 33 and 39). Subsequent linear regression analyses conducted on the data used by Vijayvergiya and Ghazzaly also do not produce this parallelism.[4]

Although the family of lines generated by linear regression analyses do not exhibit the anticipated parallelism, there does exist a degree of parallelism in each family (Figs. 33 and 39) which suggests that a closer examination might indeed reveal a trend. Thus, each range of data points is examined further as follows:

1) The value of the correlation coefficient (r) for each range of data points in each soil category is examined, and the range which has the largest absolute value of r is chosen as the base line for that family of lines. This is done because the nearer the absolute value of r is to unity,

[4]Plots of linear regression analysis done on the data used by Vijayvergiya and Ghazzaly are not included in this report.

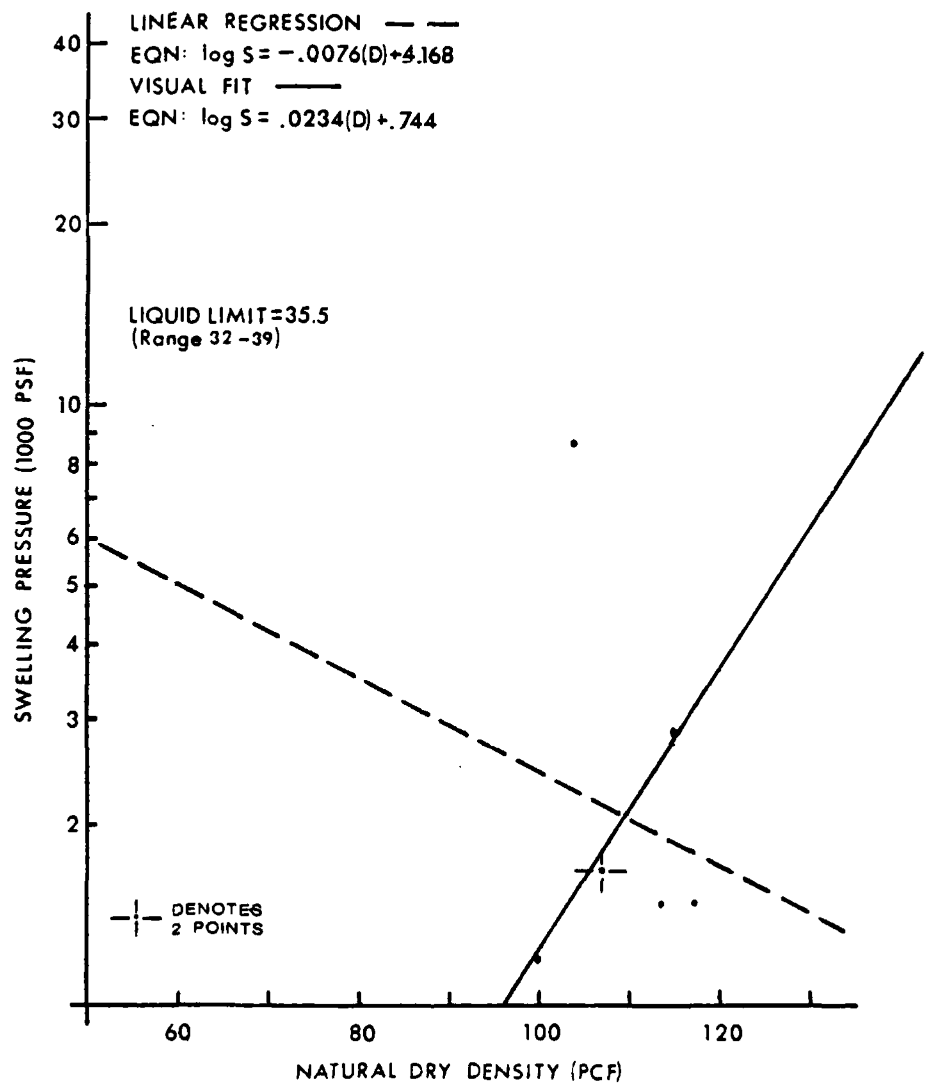


Figure 28

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Claystones)

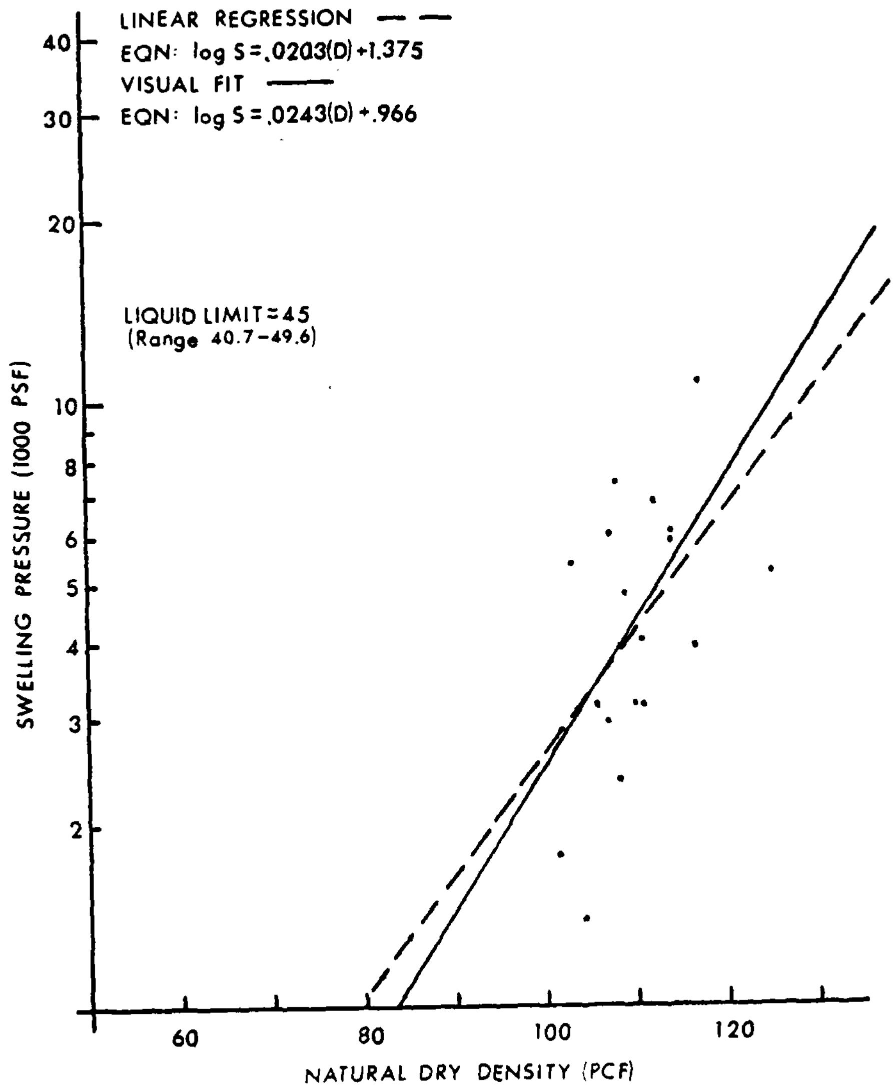


Figure 29

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Claystones)

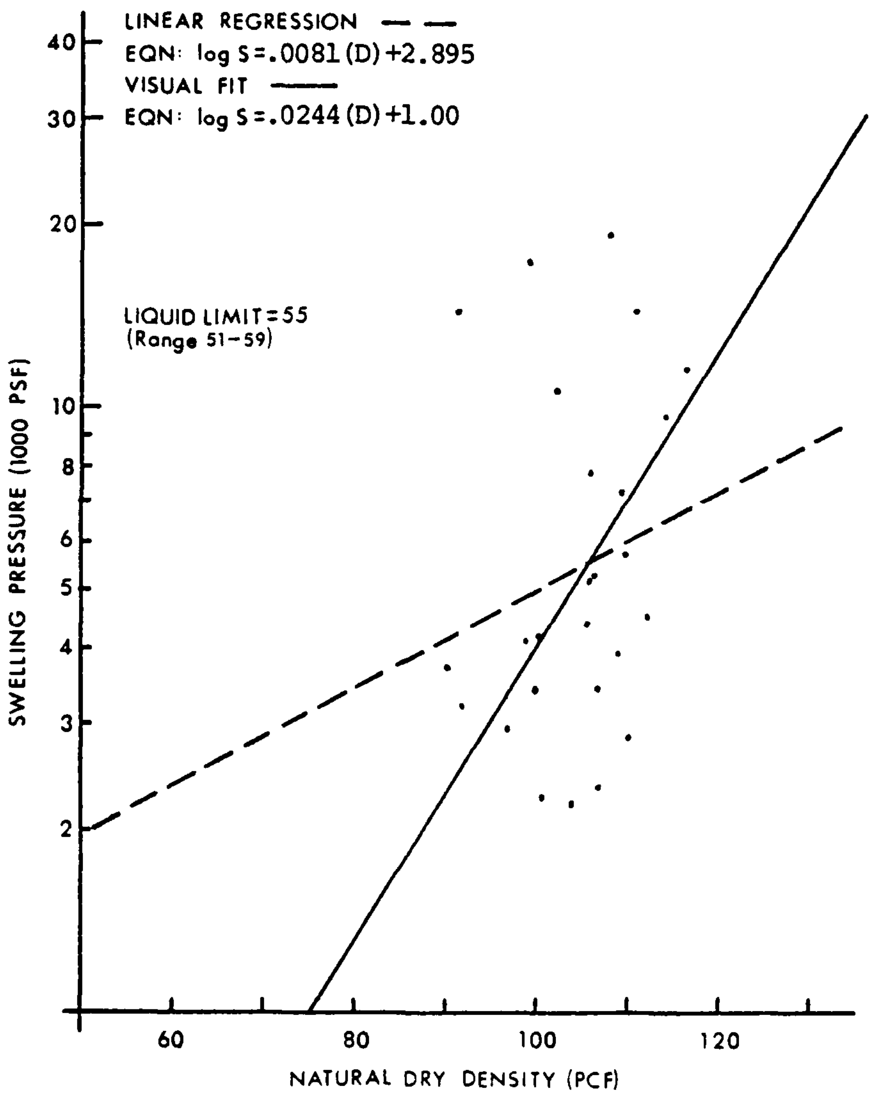


Figure 30

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Claystones)

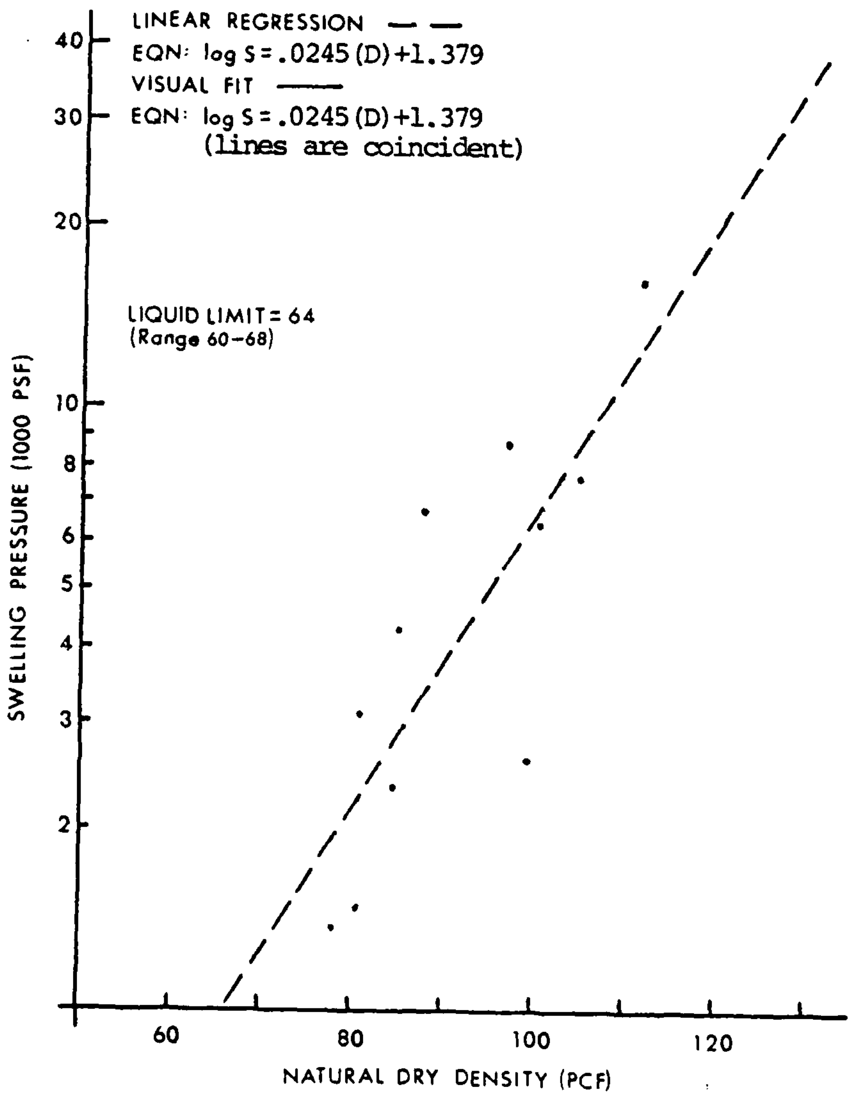


Figure 31

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Claystones)

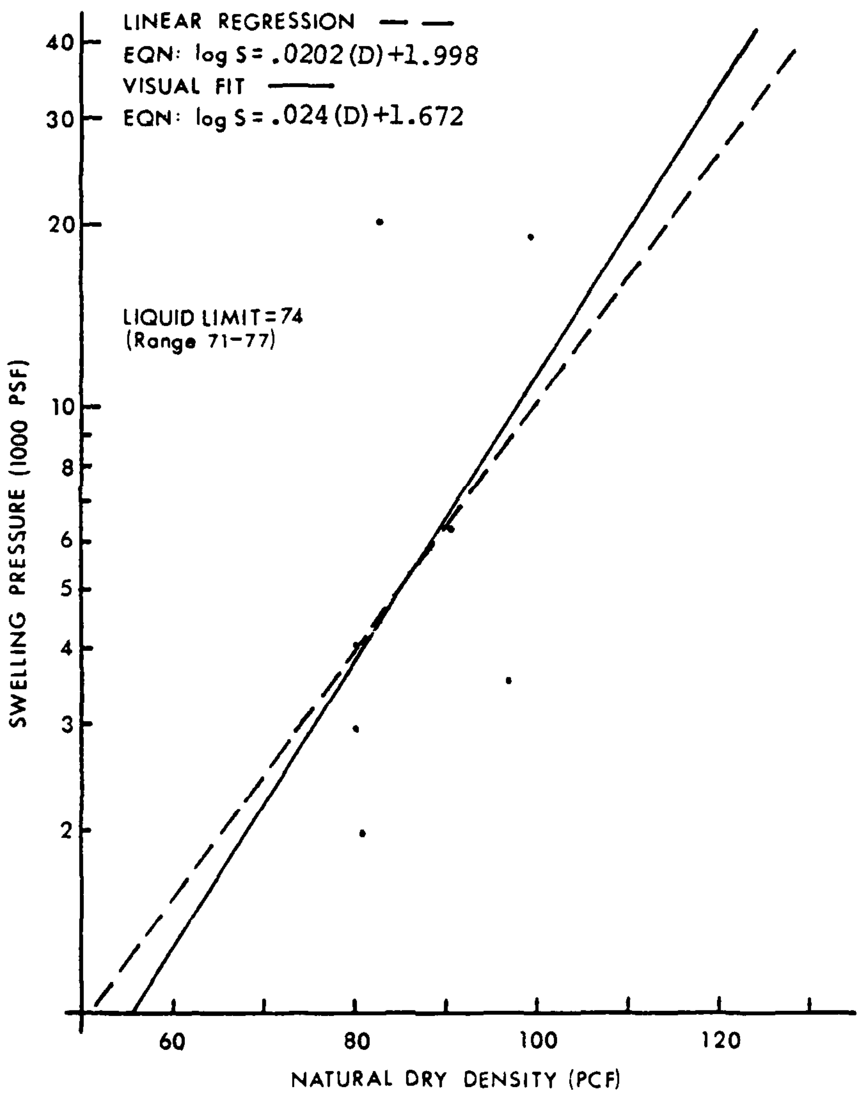


Figure 32

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Claystones)

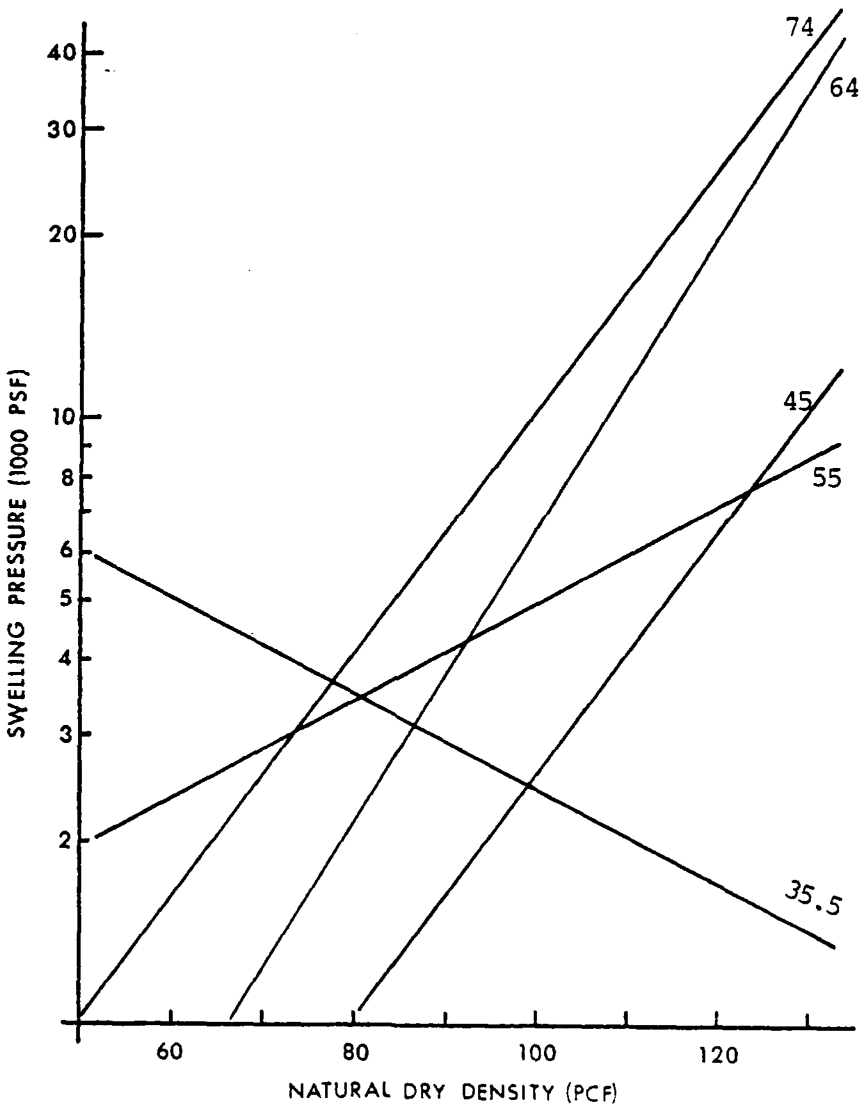


Figure 33

Family of Linear Regression Lines for
Correlation of Swelling Pressure with
Natural Dry Density and
Liquid Limit (Claystones)

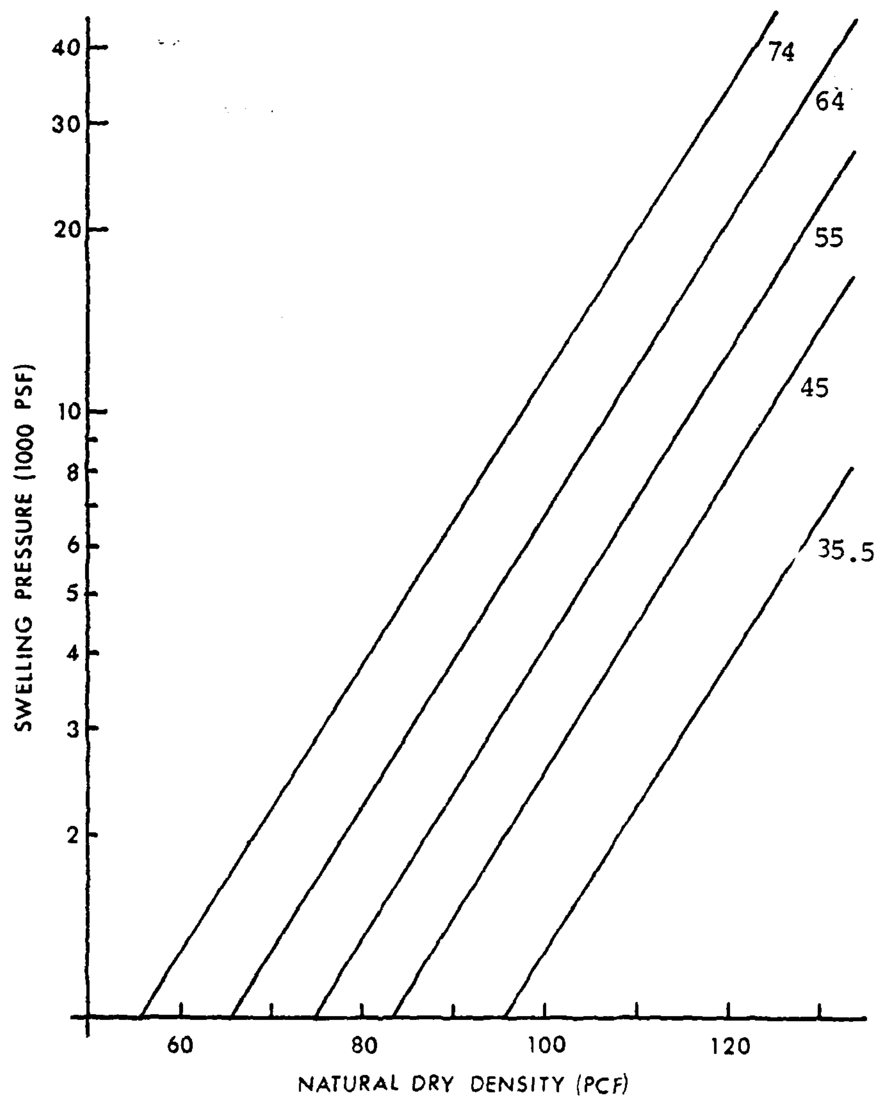


Figure 34

Family of Visually Fitted Lines for Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Claystones)

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A MEANS OF PREDICTING SWELLING PRESSURES OF SOILS FOUND IN THE --ETC(U)
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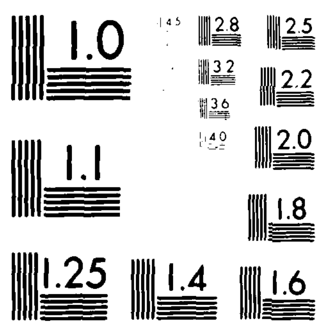

MICROCOPY RESOLUTION TEST CHART

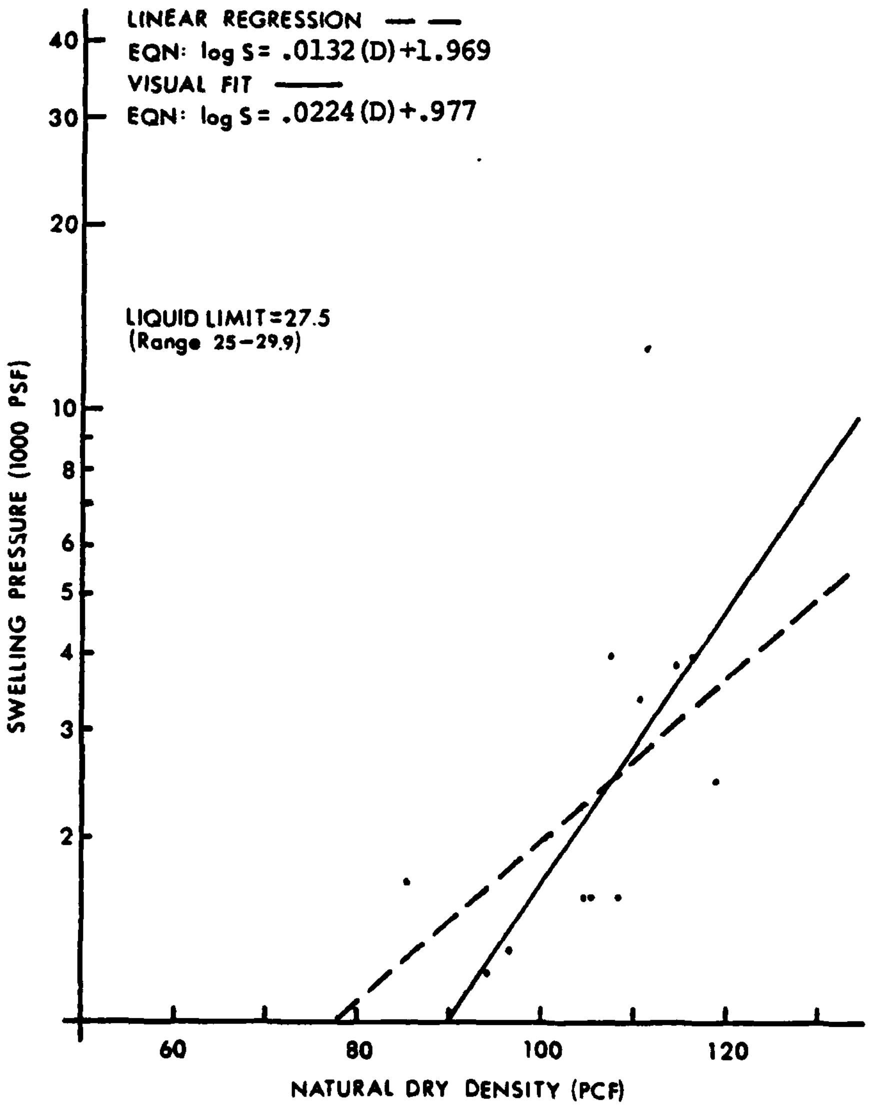

Figure 35

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Clays)

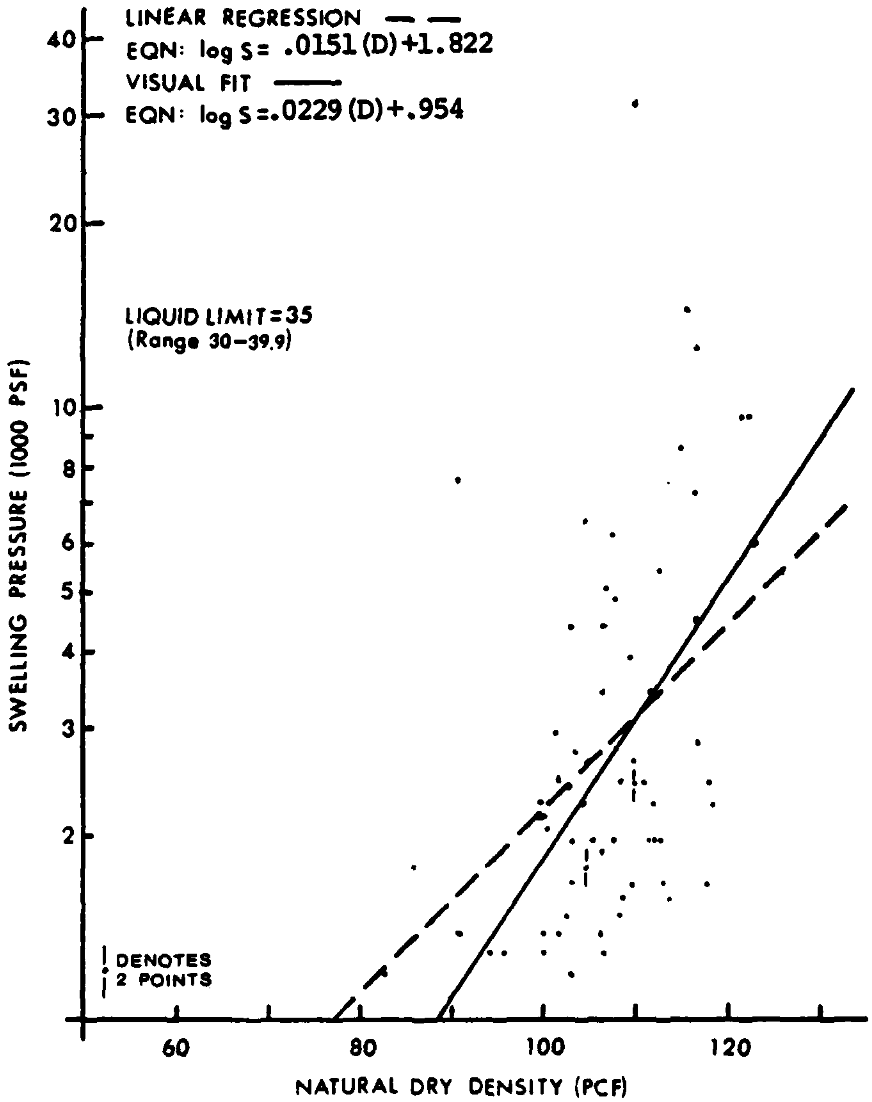

Figure 36

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Clays)

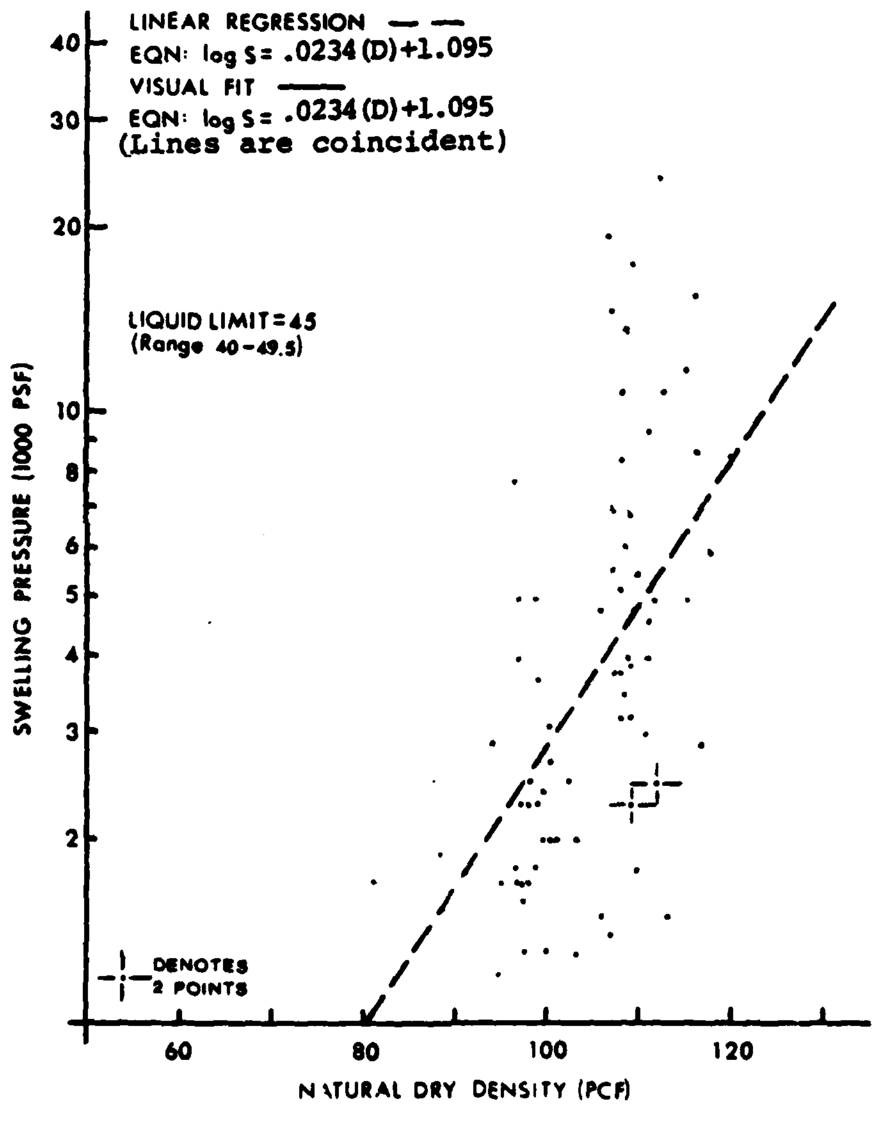

Figure 37

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Clays)

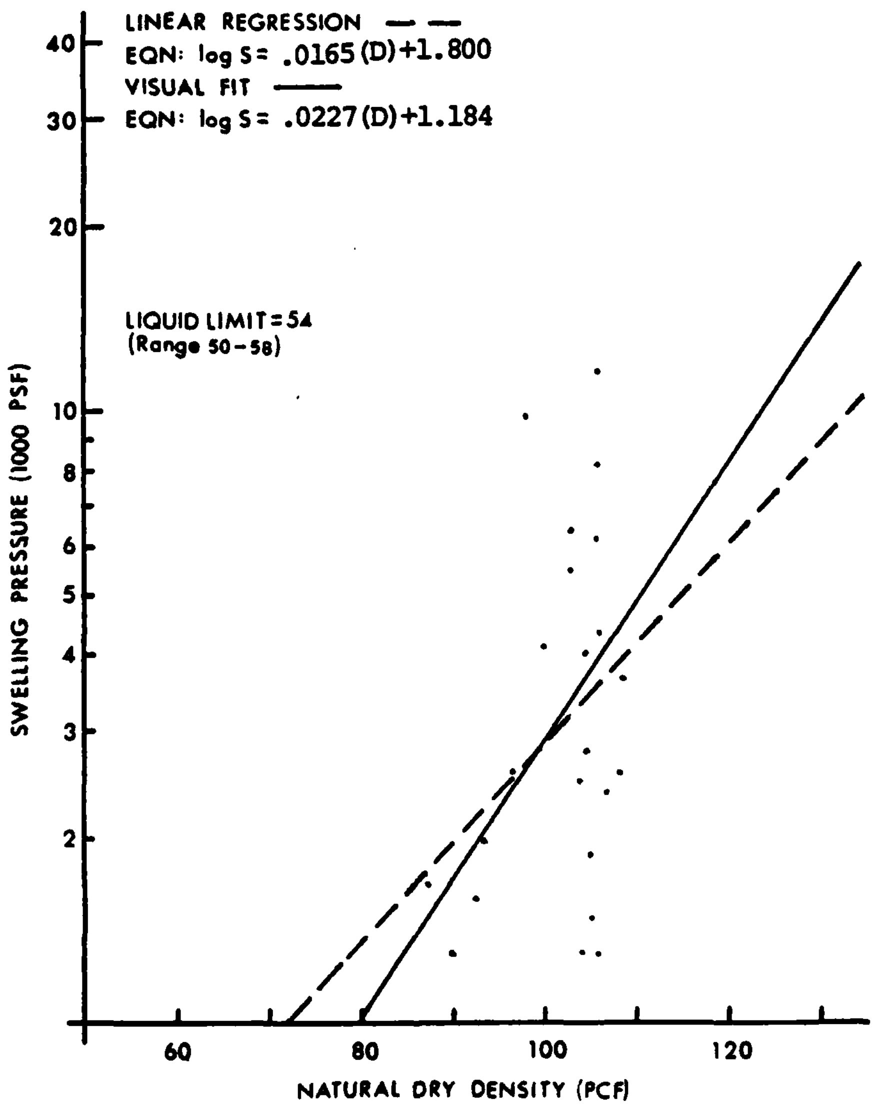

Figure 38

Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Clays)

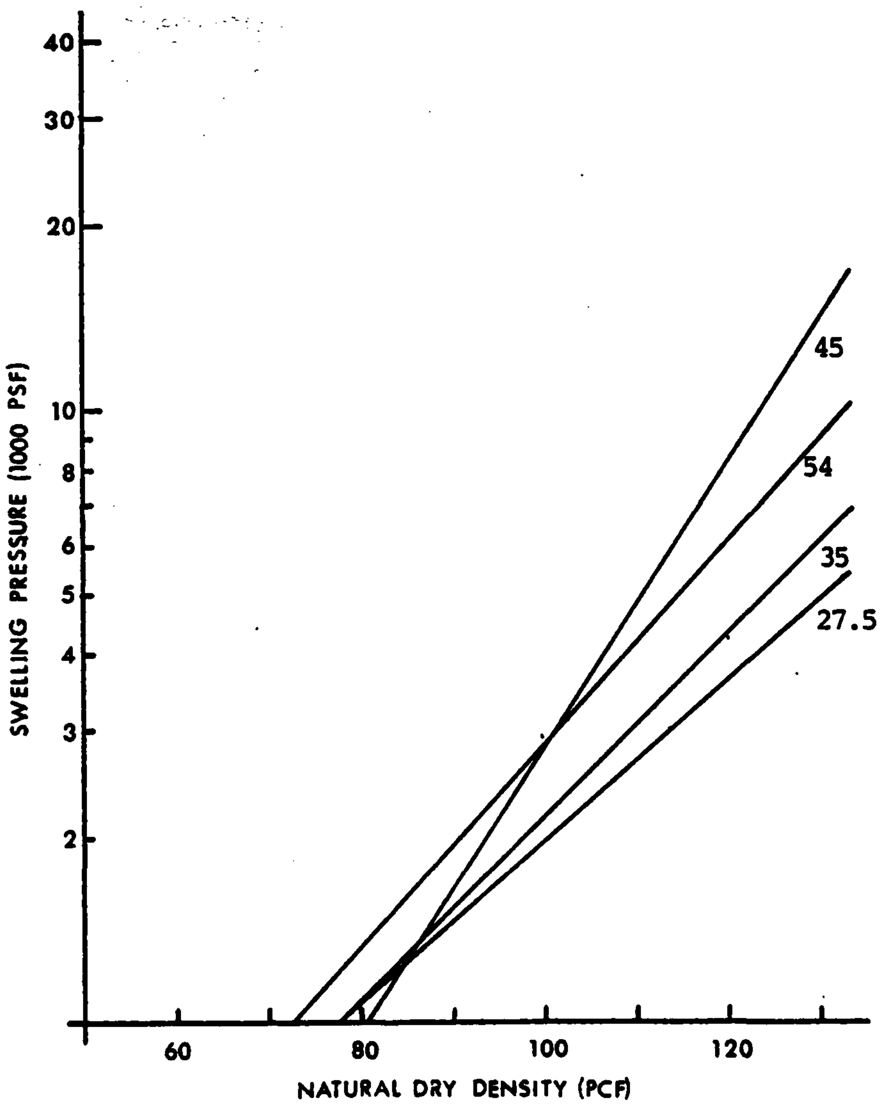

Figure 39

Family of Linear Regression Lines for Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Clays) the more closely the data scatter pattern may be represented by a straight line. For clays, for example, the line for the liquid limit range 40-49.5 is selected as the base line since the r value for the data in this range is equal to .53. Table 12 gives the values of r for each range of data.

2. Next, the r values of the other data ranges are examined. The lower that the absolute values of r are, the less linear is the inherent relationship between the data points of any particular range. Likewise, the lower that any value of r is, the less is the significance of the corresponding linear regression line as the "best fit" for that particular range of data. Consequently, other equally good fitted lines might exist.

3. By trial and error, other "good fit" lines are matched to the data points in each range. The number of data points which comprise the particular range of data, and the magnitude of the absolute value of r are taken into account during this process. The larger that the number of data points and the value of r are, the less leeway is allowed in the adjustment of any

Table 12

RESULTS OF ANALYSES

Soil Type	Liquid Limit (Range)	PI/-200 (Range)	Correlation Coefficient (r)	Equation of Linear Regression Line	Equation of Visually Fitted Line
Clay	25-29.9		.47	Log S =.0132(D)+1.969	Log S =.0224(D)+ .977
Clay	30-39.9		.40	Log S =.0151(D)+1.822	Log S =.0229(D)+ .954
Clay	40-49.5		.53	Log S =.0234(D)+1.095	Log S =.0234(D)+1.095
Clay	50-58		.34	Log S =.0165(D)+1.800	Log S =.0227(D)+1.184
Clay		.20-.29	.36	Log S =.0129(D)+2.126	Log S =.0324(D)+ .056
Clay		.30-.39	.35	Log S =.0158(D)+1.820	Log S =.0316(D)+ .181
Clay		.40-.48	.60	Log S =.0254(D)+ .859	Log S =.0337(D)+ .013
Claystone	32-39		-.16	Log S =-.0076(D)+4.168	Log S =.0234(D)+ .744
Claystone	40.7-49.6		.52	Log S =.0203(D)+1.375	Log S =.0243(D)+ .966
Claystone	51-59		.20	Log S =.0081(D)+2.895	Log S =.0244(D)+1.000
Claystone	60-68		.80	Log S =.0245(D)+1.379	Log S =.0245(D)+1.379
Claystone	71-77		.42	Log S =.0202(D)+1.998	Log S =.0240(D)+1.672
Claystone		.21-.29	-.03	Log S =-.0011(D)+3.639	Log S =.0184(D)+1.486
Claystone		.30-.39	.13	Log S =.0027(D)+3.412	Log S =.0182(D)+1.824
Claystone		.40-.45	.10	Log S =.0035(D)+3.560	Log S +.0178(D)+2.140

Note: S = Swelling Pressure
D = Natural Dry Density

line. The minimization of residuals is also considered in the fitting process.

Figures 28 to 32 show the data points and the linear regression and visually fitted lines for each range of liquid limit for the claystones. Figures 35 to 38 show the same information for the clays. Figures 34 and 40 show the family of visually fitted lines for claystones and clays respectively and may be utilized to predict the swelling pressure for the appropriate materials.

The second analysis is similar to the first except that the parameter PI/-200 is used as the discriminating parameter instead of the liquid limit value. The analysis procedure described above is used again to generate the appropriate lines, and Figures 41 to 50 reflect these results.

E. Conclusions

Using empirical data, equations have been deduced which allow for the prediction of swelling pressures for Rocky Mountain area soils. Data for claystones and clays have been analyzed by two methods, resulting in varying degrees of success. The first method correlates swelling pressure to the natural dry density and liquid limit of a soil. This method produces better results for both claystones and clays. Figure 34 shows that for claystones a good relationship exists between these variables. The

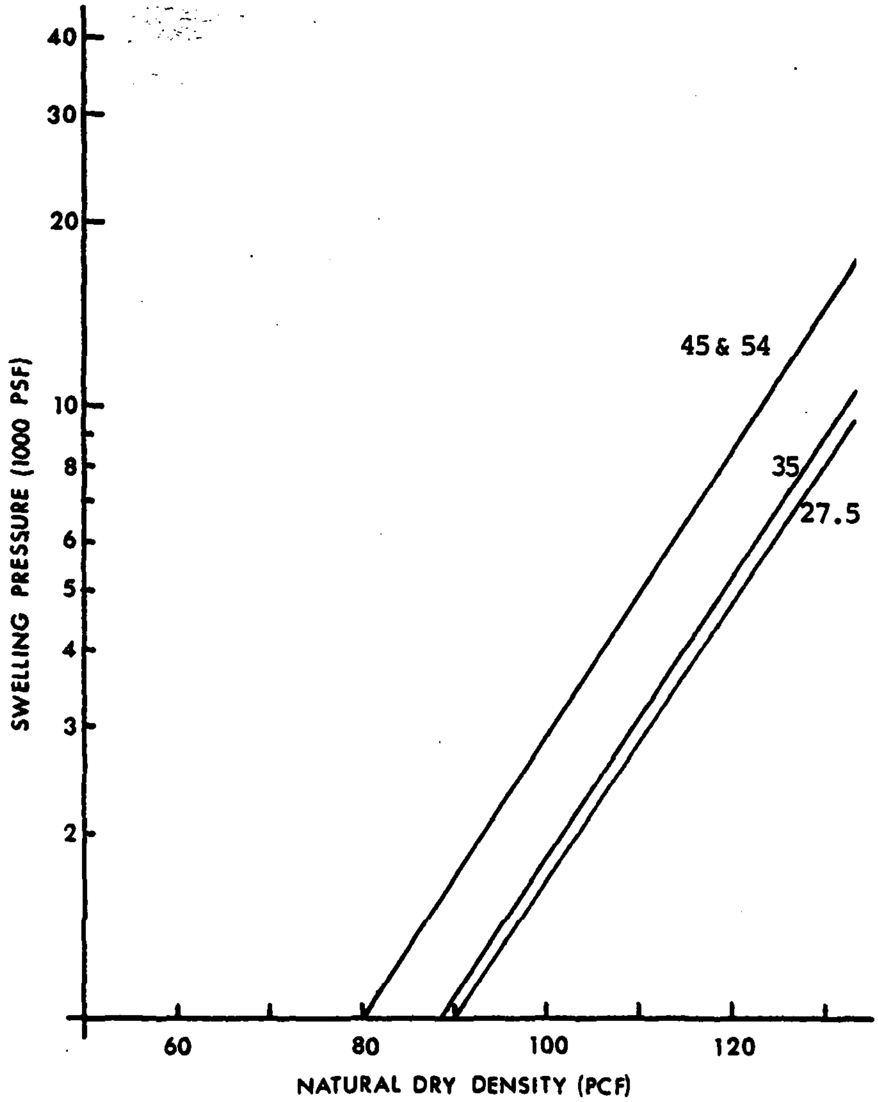

Figure 40

Family of Visually Fitted Lines for Correlation of Swelling Pressure with Natural Dry Density and Liquid Limit (Clays)

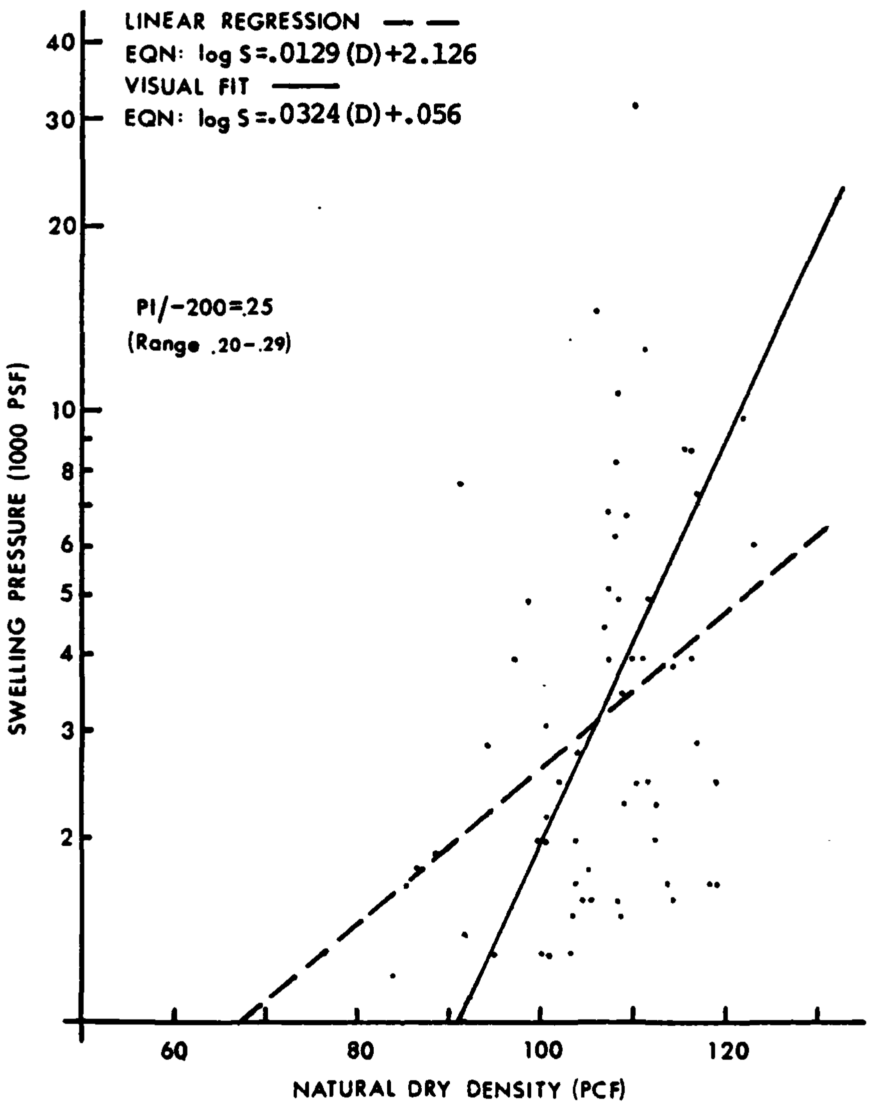

Figure 41

Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Clays)

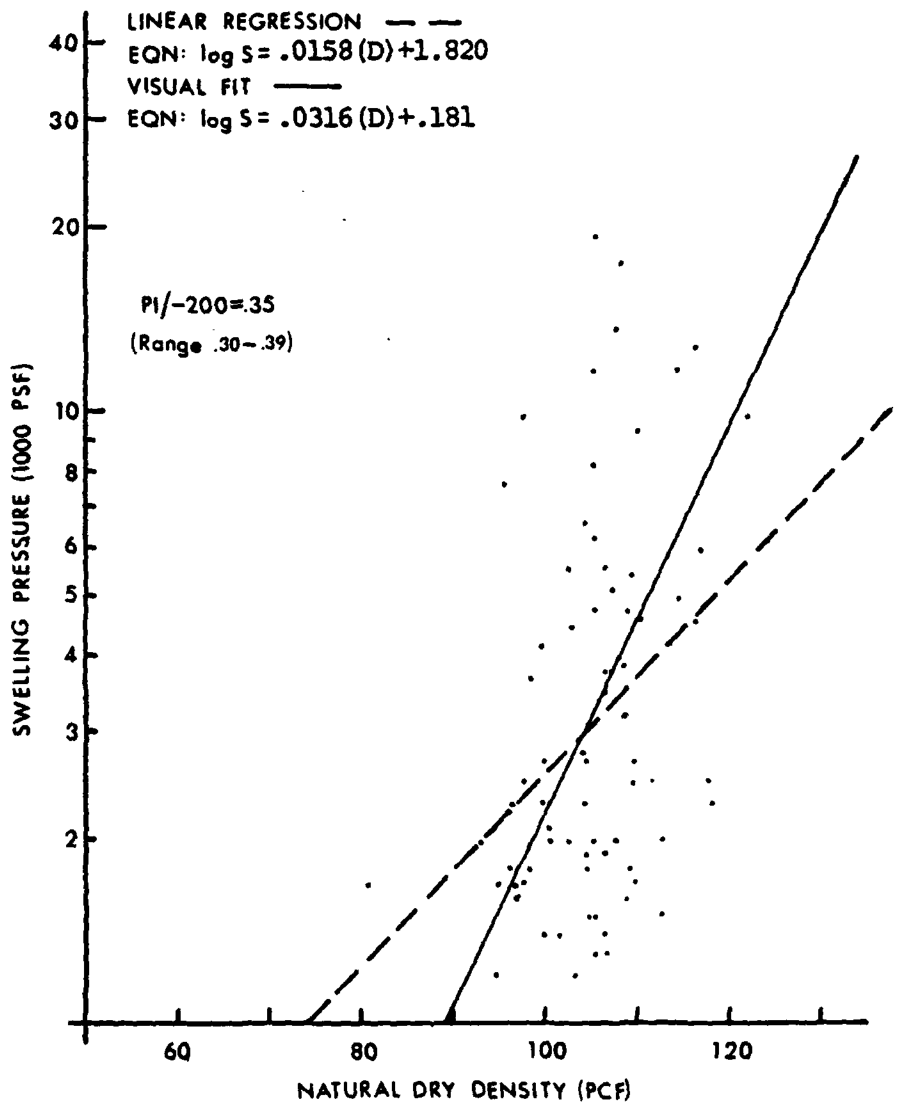

Figure 42

Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Clays)

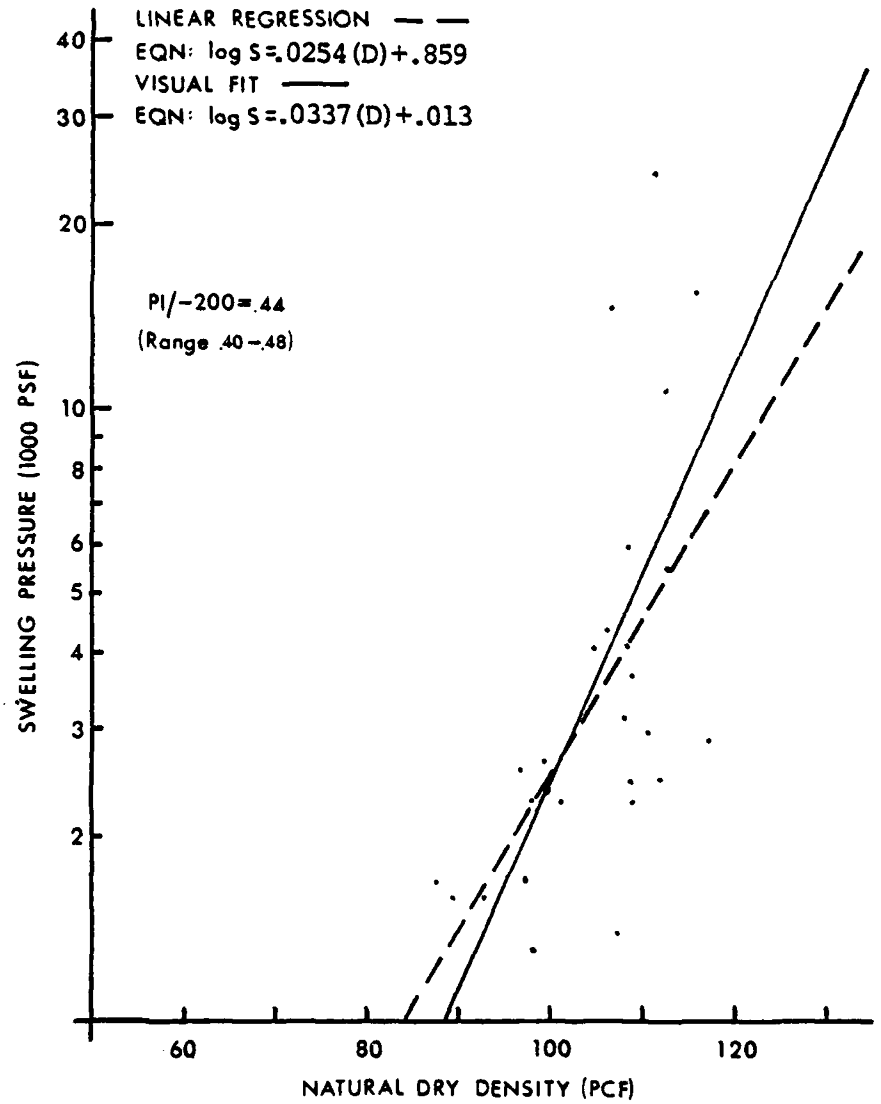

Figure 43

Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Clays)

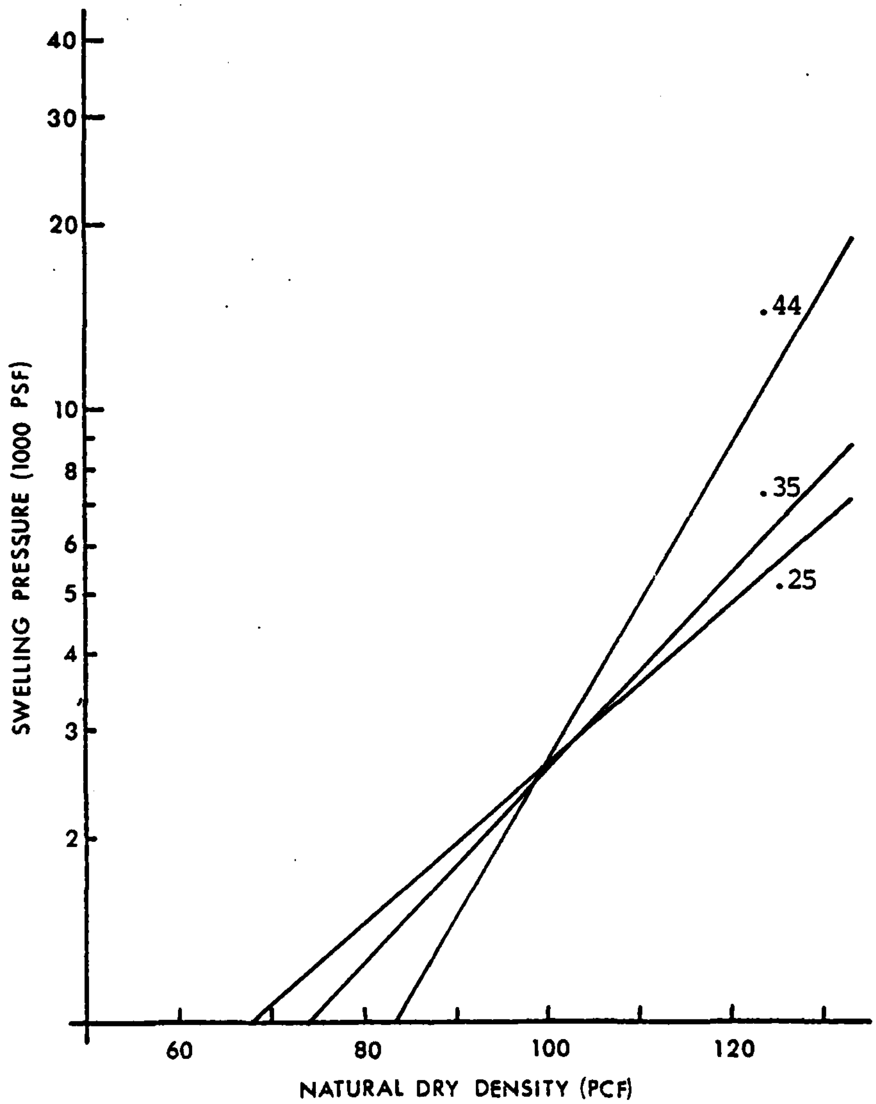

Figure 44

Family of Linear Regression Lines for Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Clays)

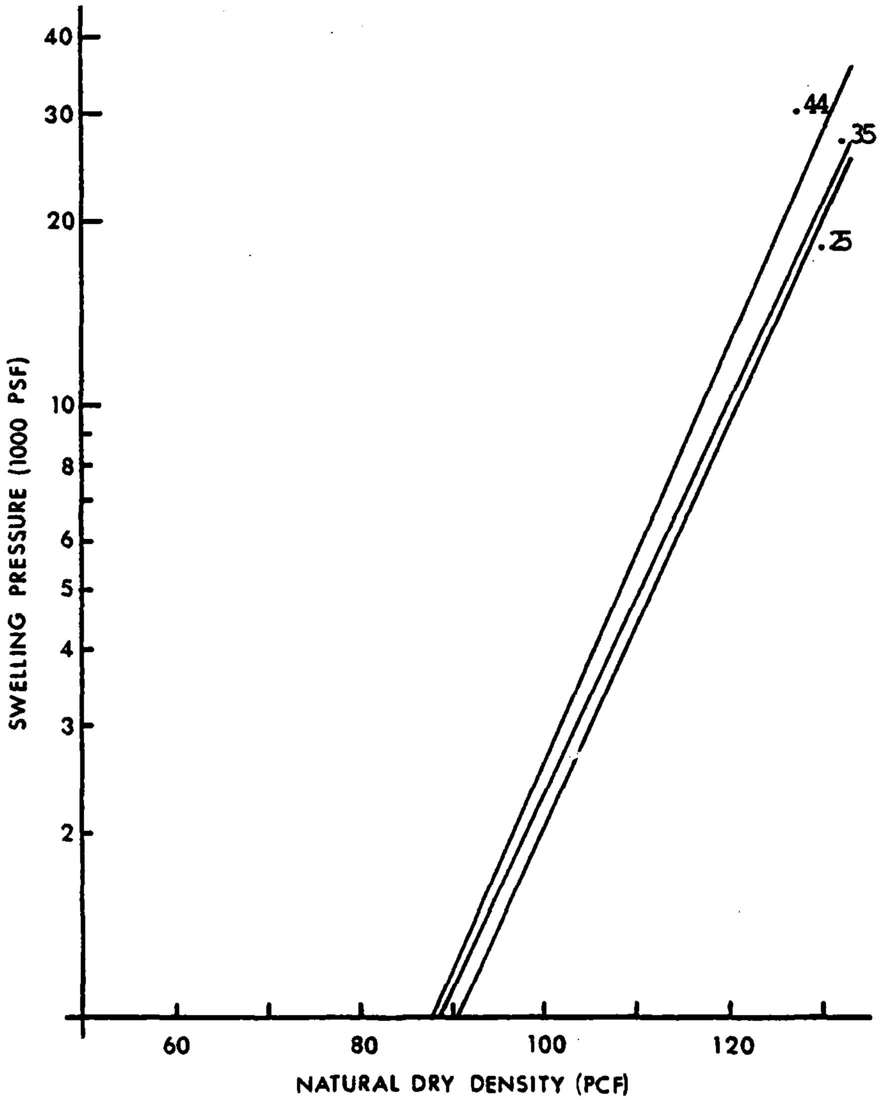

Figure 45

Family of Visually Fitted Lines for
Correlation of Swelling Pressure with
Natural Dry Density and
PI/-200 (Clays)

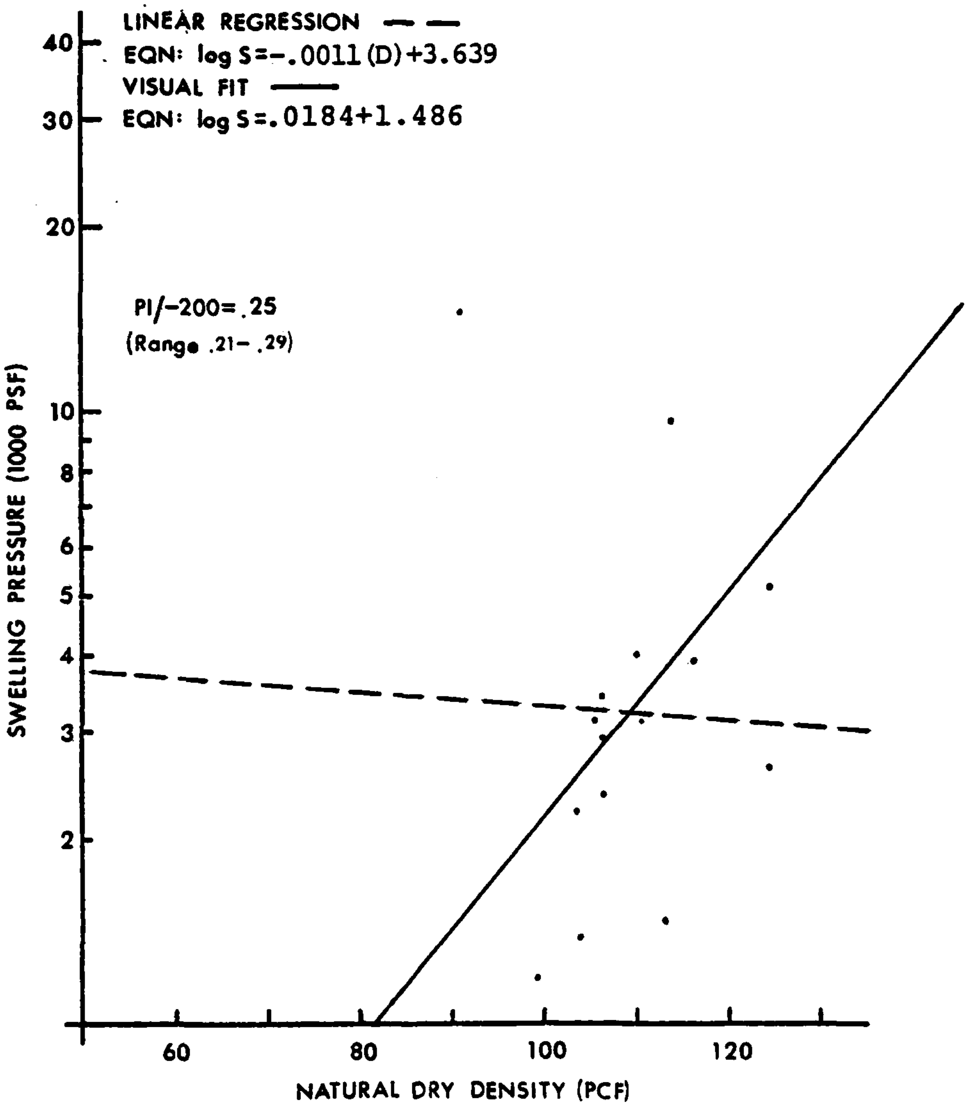

Figure 46

Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Clays)

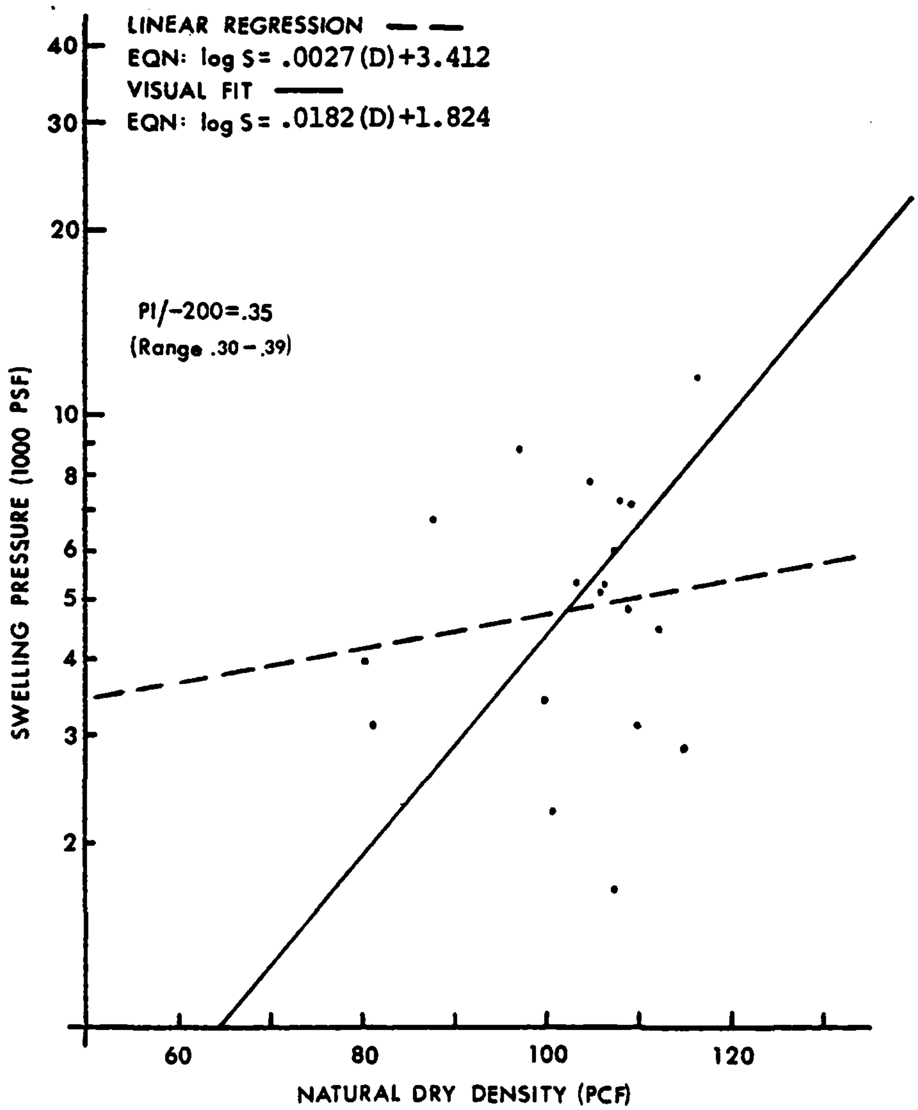

Figure 47

Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Claystones)

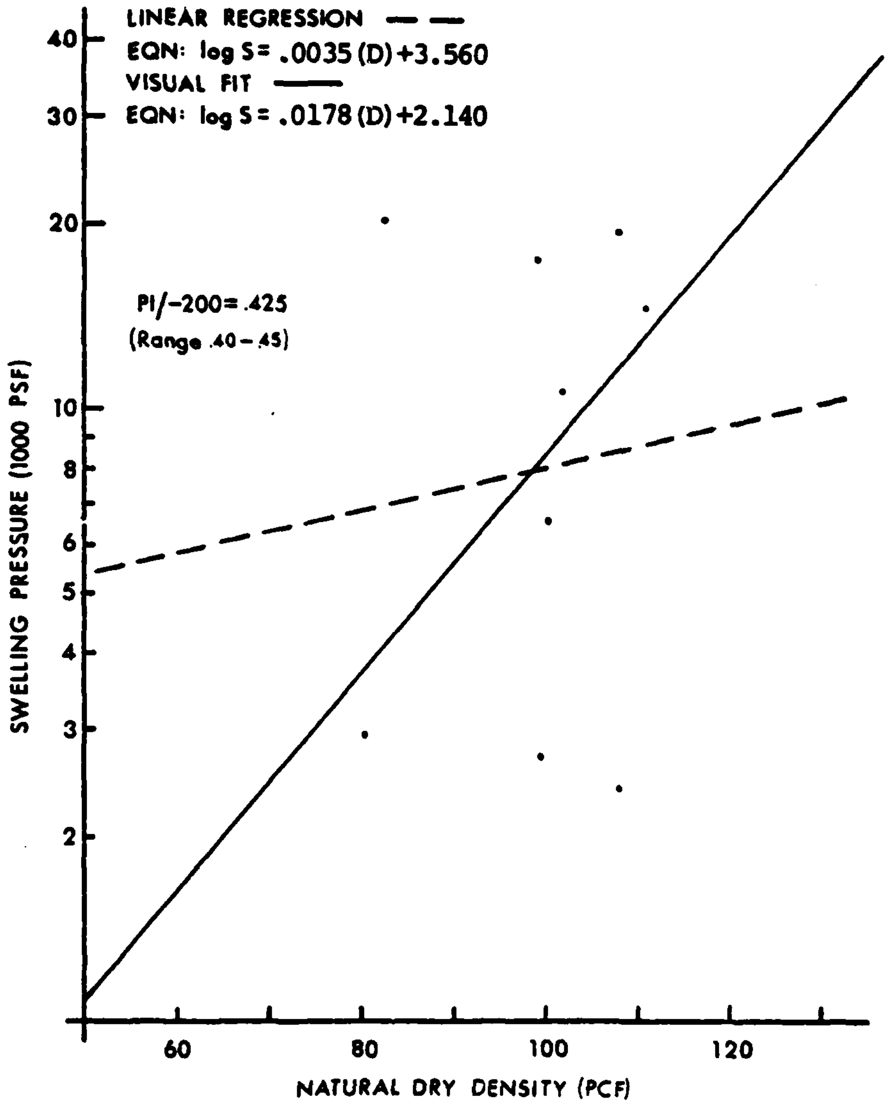

Figure 48

Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Claystones)

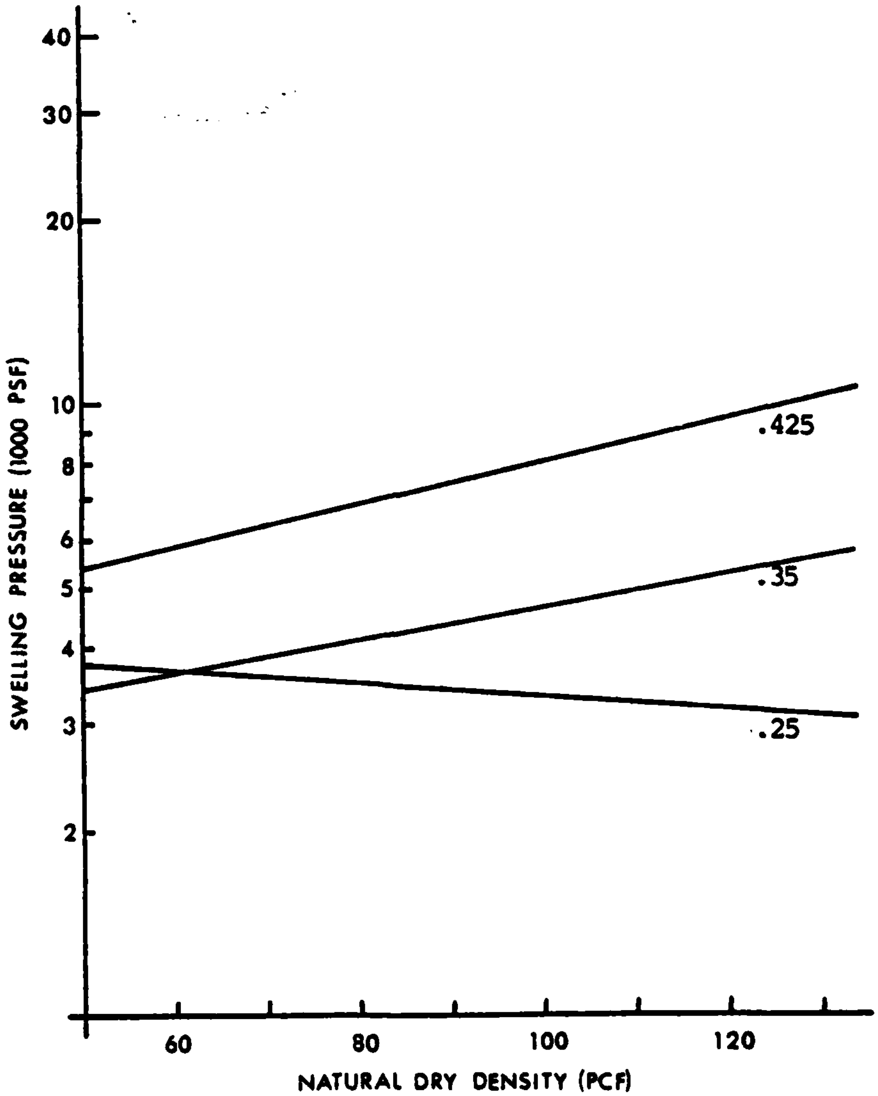

Figure 49

Family of Linear Regression Lines for
Correlation of Swelling Pressure with
Natural Dry Density and
PI/-200 (Claystones)

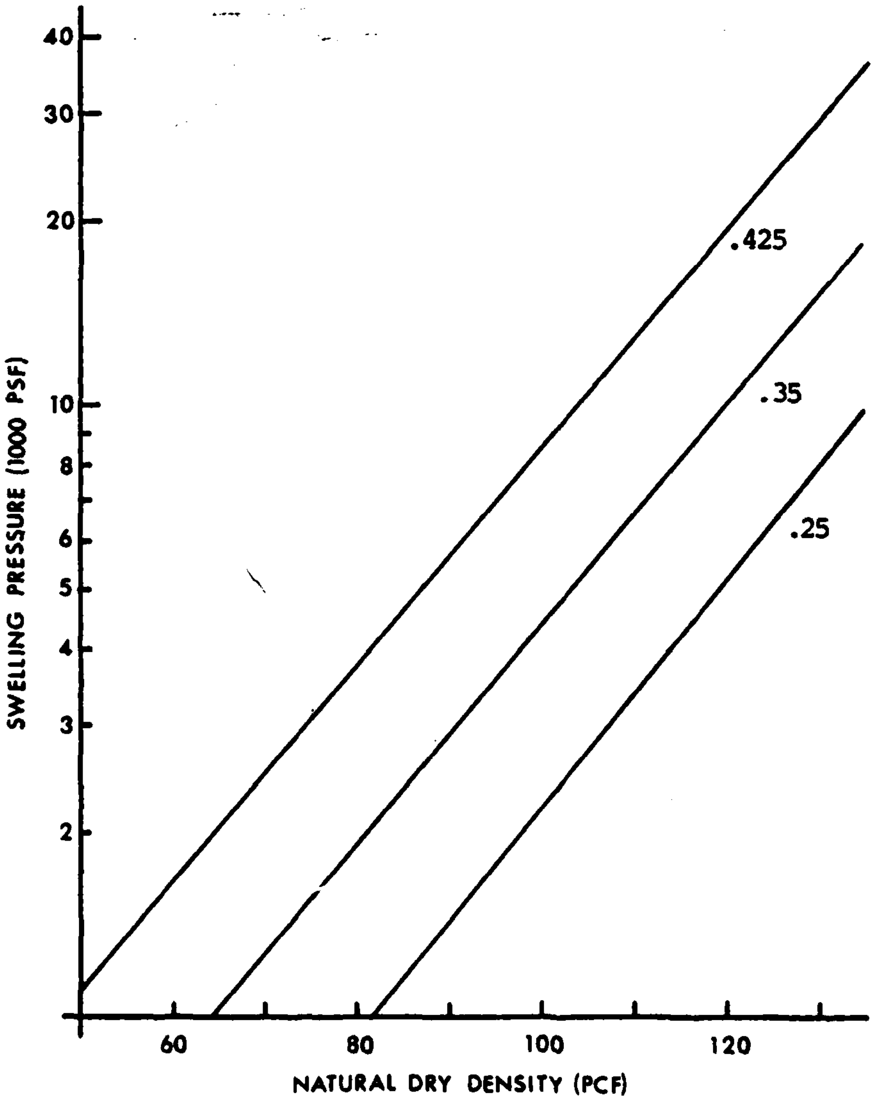

Figure 50

Family of Visually Fitted Lines for Correlation of Swelling Pressure with Natural Dry Density and PI/-200 (Claystones) swelling pressure of the claystone samples increases as the dry density increases. Also, for a given dry density, an increase in swelling pressure occurs as the liquid limit value increases. A similar relationship exists for clay soils, however, the effect of liquid limit for these soils is not as clearly defined as it is for claystones. Figure 40 shows that the swelling pressure for clays increases as the dry density increases, but that for a given dry density, the effect that an increase in the liquid limit has upon the swelling pressure is not as great. The lines for liquid limit values of 45 and 54 (ranges 40 to 49.5 and 50 to 58) are essentially coincident. Similarly, the lines for liquid limit values 27.5 and 35 (ranges 25 to 29.9 and 30 to 39.9) are also nearly coincident.

The reasons why claystones exhibit a more discernable trend than do clays is not readily attributable to experimental error. As stated earlier, all data values were accepted as reported. No effort was made to determine if testing inconsistencies existed. However, it is felt that since the data is randomly selected and encompasses several years of record, variations in test procedures would not be a major factor. Even if variations occurred, statistically the same variation should apply to the clays as well as to the claystones. The inherent experimental error associated with the determination of liquid limit or dry density values could contribute to the scatter of data points, but should balance out for large data bases and not affect the overall trend of the line. The only other explanation for the more predictable results of claystones seems to be that claystones are less prone to disturbances during sampling than are clays. Thus, for the given test procedures and analysis method used, Rocky Mountain claystones seem to swell more predictably than do clays.

The second evaluation method correlates swelling pressure to dry density and the parameter PI/-200. For both claystones and clays, this method is of less value than the first method. Although Figure 50 shows a nice trend for claystones, less confidence is placed in this family of curves than for those lines obtained by correlating swelling pressure to liquid limit. This is so because for each range of data grouped according to the value of PI/-200, the correlation coefficient is significantly lower (see Table 12). In the case of clays, Figure 45 shows a nearly coincident family of lines for data grouped using this parameter. Thus, the discrimination afforded by the parameter PI/-200 is of little practical value.

The analysis shows that swelling pressures may be predicted for Rocky Mountain soils using the method of Vijayvergiya and Ghazzaly. The results obtained with claystones seem to be better than those obtained for clays. It should be kept in mind that since the soils examined represent many locations, that the deduced equations (Table 12) can only yield estimates of typical swelling pressures. Appendix B shows plots of calculated versus measured values of swelling pressure for both claystones and clays which were analyzed by using the liquid limit parameter. Examination of these plots shows that the deduced equations generally produce swelling pressure estimates larger than measured. In those instances where the equations yield low estimates, the swelling pressures involved are of such magnitude (i.e. $\geqslant$ 4000 psf) that design measures, such as pier foundations, will be required in any event.

It is hoped that the equations shown in Table 12 will be of some practical use to soil engineers in this geographical region.

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APPENDIX A

SUMMARY OF LABORATORY TEST RESULTS

Table A-1

MMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY — (PCF)	ATTERBERG LIMITS		-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)					
16935/8	4.0	16.0	110.9	25	10	85	0.7	3400	.12	Winter Park, CO
16850/39	3.0	7.3	96.9	25	11	89	0.5	1300	.12	Montezuma Cnty, CO
16410/4	3.0	6.4	94.3	26	12	65	0.5	1200	.18	Denver, CO
17413A/18/1	7.0	9.2	105	26	12	54	0.4	1600	.22	Thornton, CO
16585/1	4.0	8.7	108.7	28.1	15.4	53	0.7	1600	.29	Aurora, CO
16577/1	4.0	14.6	115.0	28.6	14.4	68	0.6	3900	.21	Greenwood Village, CO
16638/D38	2.0	16.6	105.6	29	16	61	0.3	1600	.26	Brighton, CO
16900/5	3.0	13.3	116.9	29	16	58	0.7	4000	.28	Thornton, CO
16940/2	9.0	12.1	119.3	29	14	52	0.3	1700	.27	Thornton, CO
16940/4	4.0	8.4	111.6	29	16	61	4.5	13000	.26	Thornton, CO
17799/1	8	11.8	119.3	29	14	63	0.5	2500	.22	Jefferson Cnty, CO
17624/1	4.0	8.1	85.3	29.3	13.6	68	1.1	1700	.20	Colorado Sprgs, CO
16701/1	4.0	9.8	107.9	29.9	19.2	67	2.5	4000	.29	Jefferson Cnty, CO
9189/1	4.0	13.3	118.9	30	14	61	0.4	1700	.23	Colorado Sprgs, CO
9200/6/2	4.0	15.2	112.8	30	12	65	0.5	3500	.18	Arapahoe Cnty, CO
16763/3	39.0	26.0	96.6	30	10	65	0.2	1300	.15	Denver Cnty, CO
16900/7	3.0	11.8	123.5	30	18	52	4.0	1000	.35	Thornton, CO
16510/27	4.0	16.9	110.9	30	17	56	0.2	1700	.30	Routt Cnty, CO
17499/1	2.0	10.4	105.4	30	16	66	0.6	1800	.24	Aurora, CO
17544/5	2.0	10.2	112.8	30.5	13.2	78	0.5	2000	.17	Durango, CO
16850/5	4.0	16.7	110.8	31	19	70	7.5	33000	.27	Montezuma Cnty, CO
17735/4	13.0	14.9	102.2	31	10.3	96.9	0.5	3000	.11	Glenwood Sprgs, CO
17509/23	2.5	14.6	114.9	32	16	80	1.0	1600	.20	Denver, CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
16834/6	2.0	11.8	117.4	32	19	76	1.7	7500	.25	Lafayette, CO
16517/8	2.0	12.9	119.0	32	18.5	50	0.6	2500	.37	Jefferson Cnty, CO
16509/1	4.0	20.1	103.6	32	17	79	0.3	1500	.22	Routt Cnty, CO
17413A7/2	8.0	9.2	119.3	33	20	52	1.3	2300	.38	Thornton, CO
17533/2	7.0	16.8	114.1	33	14	65	0.3	1700	.22	Broomfield, CO
16850/Pit6	1.5	10.4	113.0	33	15	75	0.6	2300	.20	Montezuma Cnty, CO
16817/10	4.0	13.3	100.9	33	19	74	0.4	2200	.26	Westminster, CO
16608/1	4.0	18.7	104.4	33.3	21.0	86	0.9	2800	.24	Denver, CO
16550/2	2.0	13.8	106.6	33.9	18.2	73	4.2	15000	.25	Colorado Sprgs, CO
17842/5	4.0	7.5	107.2	34	19	50	0.7	1900	.38	Arapahoe Cnty, CO
16638/D33	3.0	21.7	101.0	34	19	76	0.2	1300	.25	Brighton, CO
16551/2	2.0	9.3	105.8	34	20	57	0.9	1800	.35	Adams Cnty, CO
16452/10	4.0	15.1	91.6	34	18	73	0.4	1400	.25	Littleton, CO
16446/6	4.0	16.1	108.6	35	20	55	0.5	2000	.36	Denver, CO
17413A/5/1	3.0	13.6	113.9	35	19	52	0.8	2000	.37	Thornton, CO
17453/2	4.0	11.8	104.1	35	17.1	77		2000	.22	Douglas Cnty, CO
17794/1	4.0	9.8	107.2	35	18	55	0.7	1400	.33	Adams Cnty, CO
9200/14/2	9.0	10.6	104	35	19	75	0.7	1700	.25	Arapahoe Cnty, CO
16935/9	2.0	18.2	108.4	35	17	72	1.8	6400	.24	Winter Park, CO
16452/6	3.0	17.6	105.6	35	22	69	0.3	2700	.32	Littleton, CO
16510/34	4.0	16.8	111.9	35	22	79	0.7	2500	.28	Routt Cnty, CO
17451/9	6.5	12.3	107.5	35.4	21.6	57		3500	.38	Denver, CO
17544/1	8.0	11.2	123.8	35.4	21.7	84	2.9	6200	.26	Durango, CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
16404/1	4.0	10.6	104.0	35.6	21.5	57	2.0	4500	.38	Westminster, CO
17544/8	2.0	10.0	105.5	35.9	22.6	72	2.7	6700	.31	Durango, CO
16404/3	3.0	10.8	106.2	36	16	52	1.0	2000	.31	Denver, CO
16427/2	4.0	13.4	117.7	36	19	58	1.0	4600	.33	Denver, CO
16576/2	3.0	10.8	91.5	36	20	90	2.7	7800	.22	Arapahoe Cnty, CO
9200/1/4	4.0	10.0	95.1	36	18	63	0.5	1300	.29	Arapahoe Cnty, CO
9201/4/4	4.0	18.4	110.9	36	16	67	0.4	2500	.24	Arapahoe Cnty, CO
16785/3	4.0	11.5	117.9	36	22	50	0.8	2900	.44	Thornton, CO
16900/3	3.0	9.9	109.9	36	23	63	0.7	1600	.37	Thornton, CO
16576/1	4.0	10.8	101.2	36	20	64	0.7	2100	.31	Arapahoe Cnty, CO
16386/6	2.0	9.7	107.6	36.5	22.7	67	0.3	1300	.34	Westminster, CO
16596/1	3.0	10.8	122.7	36.7	23.3	90	5.8	10000	.26	Lamar, CO
16324/4/4	4.0	10.4	83.6	37	18	92	0.3	1200	.20	Denver, CO
17444/15	4.0	10.7	116.0	37	18	64		8900	.28	Aurora, CO
17542/5	4.0	13.5	113.7	37	23	57	2.7	5600	.40	Adams Cnty, CO
17842/21	3.0	14.0	110.2	37	17	65	1.3	4000	.26	Arapahoe Cnty, CO
16595/Pit2	4.0	7.7	102.4	37	20	92	1.3	2500	.22	Cortez, CO
9184/5	3.0	17.4	107.9	37	14	54	1.2	5200	.26	Arapahoe Cnty, CO
16817/1	4.0	11.9	117.9	37	24	74	3.6	13000	.32	Westminster, CO
16900/2	3.0	18.9	105.2	37	21	61	0.5	2300	.34	Thornton, CO
16494/1	4.0	19.8	107.3	37	21	91	1.3	4500	.23	Steamboat Sprgs,CO
16386/3	2.0	10.9	102.5	38	23.1	68	0.5	1400	.34	Westminster,CO
17801/4	2.0	10.6	104	38	21	66	0.3	1200	.32	Jefferson Cnty,CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY— (PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
17799/8	2.0	9.0	110.8	38	21	70	1.0	2500	.30	Jefferson Cnty, CO
17833/1	7.0	10.3	109.2	38	21	52	0.7	2500	.40	Inverness, CO
9200/2/3	4.0	10.7	101.7	38	21	53	1.4	2300	.40	Arapahoe Cnty, CO
16785/1	4.0	17.5	110.8	38	24	66	0.6	2700	.36	Thornton, CO
16482/4	9.0	11.1	86.6	38	23	92	0.7	1800	.25	Arapahoe Cnty, CO
17802/3	4.0	8.0	100.9	39	19	57	0.5	1400	.33	Westminster, CO
9216/1	4.0	17.9	112.7	39	20	71	0.5	2000	.28	Denver, CO
16763/1	19.0	23.7	100.7	39	23	74	0.6	2300	.31	Denver, CO
16731/2	4.0	18.8	108.8	39.5	19	98	1.2	5000	.20	Pitkin Cnty, CO
16316/1	2.0	17.6	109.2	39.9	17.6	73	0.5	1500	.24	Colorado Sprgs, CO
9193/8	8.0	11.4	98.9	40	20	78	2.7	5000	.26	Denver, CO
9229/14	18.0	14.8	116.9	40	18	74	3.5	8900	.24	Douglas Cnty, CO
16850/21	4.0	17.1	109.9	40	23	69	7.0	18000	.33	Montezuma Cnty, CO
16848/4	8.0	16.0	111.2	40	22	52	0.4	3000	.42	Denver, CO
16415/2	4.0	13.5	99.4	40.3	19.7	61.5	1.7	3700	.32	Denver, CO
17477/6	3.0	20.4	109.6	40.3	21.2	96		2300	.22	Pueblo, CO
16942/4	3.0	15.5	112.4	41	25	60	0.8	2500	.42	Federal Heights, CO
16735/21	24.0	16.5	100	41	24	57	0.3	2400	.42	Jefferson Cnty, CO
16848/1	9.0	17.9	108.3	41	22	71	1.9	5200	.31	Denver, CO
16488/6	3.0	11.5	95.2	41	24	73	0.4	1200	.33	Arapahoe Cnty, CO
16385/3	2.0	12.7	116.7	41	27	64	3.8	16000	.42	Westminster, CO
17650/20	1.5	15.1	117.2	41	23	98	1.3	2900	.23	Montrose, CO
17506/2	9.0	12.5	111.5	41	24	70	3.9	9500	.34	Adams Cnty, CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY — (PCF)	ATTERBERG LIMITS		-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)					
17808/7	3.0	16.6	112.6	41	22	60	0.3	2500	.37	Boulder, CO
16407/2	4.0	19.0	109.0	41.2	21.2	75	1.0	3500	.28	Denver, CO
9190/4	8.0	18.9	109.6	41.3	20.5	51	0.7	2300	.40	Denver, CO
9243/1	8.0	20.6	107.7	42	24	58	0.3	1400	.41	Arapahoe Cnty, CO
16817/4	2.0	13.8	118.1	42	25	78	1.8	6000	.32	Westminster, CO
16848/7	2.0	13.7	108.3	42	23	72	1.3	3800	.32	Denver, CO
16856/1	3.0	21.0	100.1	42	22	76	0.3	1300	.29	Westminster, CO
16909/1	3.0	11.7	107.4	42	24	57	4.0	15000	.42	Jefferson Cnty, CO
16946/1	4.0	15.6	109.5	42	25	89	1.5	6900	.28	Jefferson Cnty, CO
16483/6	3.0	11.6	115.7	42	26	80	2.8	12000	.33	Arapahoe Cnty, CO
17669/3	7.0	13.6	115.8	42	24	75	1.2	5000	.32	Boulder, CO
17834/3	7.0	12.6	111.5	42	22	95	1.2	4000	.23	Englewood, CO
10848/1	3.0	25.7	97.9	42.1	27.0	65.6	0.8	1700	.41	Colorado Sprgs, CO
16759/1	3.0	13.7	109.8	42.5	27.1	81	1.7	3900	.33	Arapahoe Cnty, CO
16820/2	3.0	11.5	120.7	43	28	53	1.8	8700	.53	Denver, CO
9200/6/1	4.0	12.6	96.9	43	24	76	3.4	7800	.32	Arapahoe Cnty, CO
9200/10/6	4.0	12.5	100.9	43	22	76	1.6	3100	.29	Arapahoe Cnty, CO
16900/1	3.0	13.0	110.1	43	23	68	0.6	1800	.34	Thornton, CO
16488/5	3.0	16.4	107.6	43	26	91	1.6	7000	.29	Arapahoe Cnty, CO
16488/11	4.0	14.6	103.7	43	25	88	0.5	1300	.28	Arapahoe Cnty, CO
17538/6	3.0	15.8	113.8	43	22	58	0.1	1500	.38	Lakewood, CO
17800/2	7.0	18.9	106.5	43	25	76	0.3	1500	.33	Denver, CO
16852/1	3.0	23.1	97.0	43.3	27.4	70	0.6	1800	.39	Colorado Sprgs, CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY—(PCF)	ATTERBERG LIMITS		-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)					
16473/2	3.0	11.5	97.3	44	26	96	2.2	4000	.27	Aurora, CO
16820/3	4.0	23.7	99.1	44	19	52	0.3	1800	.37	Denver, CO
16856/2	1.0	19.9	103.6	44	23	74	0.7	2000	.31	Westminster, CO
16898/1	3.0	11.7	97.2	44	27	50	2.5	5000	.54	Lakewood, CO
16482/1	4.0	13.8	113.1	44	30	74	3.3	11000	.41	Arapahoe Cnty, CO
17495/1	3.0	19.9	94.4	44	20	83	1.1	2900	.24	Lakewood, CO
17444/11	3.0	16.0	108.9	44	23	81		11000	.28	Aurora, CO
17602/11	3.0	10.1	95.6	44	22	64	1.0	1700	.34	Douglas Cnty, CO
16761/4	7.0	19.9	108.6	44.1	26.5	65	0.8	3200	.41	Aurora, CO
9215/4	3.0	15.5	112.1	44.2	23.3	81	2.5	5000	.29	Douglas Cnty, CO
9215/23	3.0	12.7	108.0	44.8	23.0	64.8	1.7	3800	.35	Douglas Cnty, CO
17464/22	4.0	15.1	98.3	44.9	27.5	88		1700	.31	Adams Cnty, CO
16848/11	1.0	14.0	108.7	45	27	94	3.1	8500	.29	Denver, CO
17499/11	9.0	16.1	100.0	45	26	95	0.9	2000	.27	Aurora, CO
17678/1	2.0	22.2	106.1	45	28	93	0.8	4800	.30	Englewood, CO
16483/3	2.0	12.4	110.0	46	30	87	2.3	4800	.34	Arapahoe Cnty, CO
17450/8	3.0	14.5	107.6	46	25	64		5600	.39	Golden, CO
17450/9	8.0	25.8	97.6	46	25	69		1600	.36	Golden, CO
17798/2	3.0	19.6	109.5	46	25	68	1.0	3200	.37	Lakewood, CO
16407/1	4.0	19.7	107	46.2	25.2	77	4.8	20000	.33	Denver, CO
7727/6		15.6	112.1	46.4	28.2	69.9	6.2	25000	.40	Littleton, CO
17528A/1	4.0	16.8	111.4	47	26	86	1.5	4600	.30	Boulder, CO
17689/2	2.5	14.2	109.1	47	28	86	0.6	4000	.33	Denver, CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
17538/5	4.0	25.1	98.1	47	31	72	0.2	1300	.43	Lakewood, CO
17815/1	2.0	12.8	109.0	47	28	67	0.6	6100	.42	Thornton, CO
17829/1	4.0	24	98.4	47	29	70	0.5	2300	.41	Adams Cnty, CO
9240/3	9.0	19.6	110.3	47.1	27.0	79	1.4	5500	.34	Arapahoe Cnty, CO
11979/Pit 2	2.0	23.1	100.9	47.2	26.9	75	1.2	2700	.36	Thornton, CO
16407/5	7.0	18.4	109.0	47.5	29.2	82	1.9	14000	.36	Denver, CO
12023/1	2.0	23.6	100.6	47.7	23.3	79	0.7	2000	.29	Thornton, CO
9184/1	4.0	17.0	88.9	48	22	95	1.0	1900	.23	Arapahoe Cnty, CO
9200/1/6	4.0	18.0	81.6	48	25	82	0.6	1700	.30	Arapahoe Cnty, CO
17488/16	2.0	11.6	97.4	48	25	77	1.0	1700	.32	Arapahoe Cnty, CO
17438/6	5.0	24.8	97.3	48.6	27.1	91		2300	.30	Aurora, CO
16711/21	18.0	21.5	102.8	49	29	55	1.0	2500	.53	Denver, CO
11779/24	4.0	23.2	101.4	49	25	80	0.5	2000	.31	Denver, CO
16941/6	1.0	13.8	98.4	49.5	31.7	87	1.0	2500	.36	Northglenn, CO
16443/1	9.0	23.3	100.7	50	28	94	0.8	4200	.30	Arapahoe Cnty, CO
16856/3	2.0	12.9	87.9	50	33	71	0.7	1700	.46	Westminster, CO
11979/Pit 1	2.0	16.6	106.3	50.6	27.4	86	0.3	1300	.32	Thornton, CO
17501/6	9.0	18.0	109.4	50.7	34.8	85	1.3	3700	.41	Colorado Sprngs,CO
9240/1	4.0	22.4	105.9	50.8	32	88	0.4	1500	.36	Arapahoe Cnty, CO
11979/Pit 1	4.0	22.1	105.4	50.8	27.8	81	0.6	1900	.34	Thornton, CO
17564/1	4.0	19.3	108.9	51	38	69	0.4	2600	.55	Denver, CO
17649/5	4.0	22.8	104.8	51	32	64	0.3	1300	.50	Denver, CO
9211/12	9.0	19.8	106.5	53	29	86	1.9	8400	.34	Greenwood, CO

Table A-1 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYS)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY—(PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
17501/5	2.0	6.3	105.2	53.6	34.8	93	2.5	2800	.37	Colorado Sprngs,CO
16779/1	4.0	10.9	106.4	54	31	89	2.8	6300	.35	Broomfield, CO
16531/1	4.0	25.6	97.1	54	28	58	0.7	2600	.48	Adams Cnty, CO
17714A/3	1.0	27.8	90.5	54	36	50	0.4	1300	.72	Golden, CO
16788/3	4.0	18.3	103.8	55	27	85	2.4	5600	.32	Arapahoe Cnty, CO
16531/5	2.0	21.0	105.1	55	34	86	1.3	4100	.40	Adams Cnty, CO
17403/3	4.0	17.8	106.7	55.8	33	84	3.6	12000	.39	Arapahoe Cnty, CO
12023/Pit 1	1.5	28.9	94.0	56.0	31.8	81	0.5	2000	.39	Thornton, CO
17570/2	13.0	19.2	106.7	56.0	38	81	3.8	4400	.47	Jefferson Cnty, CO
16711/1	4.0	19.4	103.7	57.0	44	86	2.8	6500	.51	Denver, CO
16460/1	3.0	19.1	98.8	57.7	34.2	99	9.0	10000	.35	Calhan, CO
17614/2	2.0	12.1	93.0	57.9	36.5	91	1.0	1600	.40	Colorado Sprngs,CO
17647/5	4.0	17.5	107.5	58	39	65	0.8	2400	.60	Littleton, CO
17836/4	9.0	22.3	104.5	58	38	71	0.5	2500	.54	Denver, CO
16480/2	2.5	29.7	89.5	60.2	37.9	94	0.3	1600	.40	Lamar, CO
16596/4	3.0	8.6	114.8	60.6	41.7	77	1.0	1800	.54	Lamar, CO
17801/2	3.0	13.6	99.9	62	39	96	2.5	2700	.41	Jefferson Cnty, CO
17763/1	2.5	26.9	93.2	74	50	98	0.9	1800	.51	Hugo Cnty, CO

Table A-2

SUMMARY OF LABORATORY TEST RESULTS (CLAYSTONES)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS		-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)					
16817/4	7.0	8.9	125.4	27	15	65	0.7	2700	.23	Westminster, CO
17533/1	4.0	14.9	114.2	32	17	82	0.1	1500	.21	Broomfield, CO
17459/2	5.0	9.9	107.6	33.5	11.1	58		1700	.19	Colorado Sprgs, CO
17438A/4	34.0	21.5	104.2	34.5	18.2	95		8900	.19	Aurora, CO
16525/2	9.0	14.3	117.6	36	10	67	0.2	1500	.15	Douglas Cnty, CO
17462/1	29.0	21.4	100.5	36	15	64		1200	.23	Denver, CO
17599/5	5.0	19.8	107.9	36	21	66	0.2	1700	.32	Golden, CO
16465/2	19.0	15.1	115.5	39	23.7	79	0.8	2900	.30	Northglenn, CO
16549/11	17.0	15.5	118.0	40.7	20		2.3	11000		Colorado Sprgs, CO
16711/22	38.0	17.5	111.3	42	21	91	1.0	3200	.23	Denver, CO
16547/1	78.0	15.0	114.8	42	21		1.5	6000		Denver, CO
10848/4	3.0	18.7	108	42.9	25.8	86.8	3.2	6200	.30	Colorado Sprgs, CO
17603/8	17.0	19.2	104.9	43	17	67	0.3	1400	.25	Frankton, CO
17793/4	4.0	16.6	109.5	44	22	57	2.1	4900	.39	Federal Heights, CO
9191/1	4.0	11.0	125.9	44	22	98.7	2.08	5300	.22	Colorado Sp[illegible], CO
16596/8	6.0	17.1	108.7	44.6	27	79	2.7	7500	.34	Lamar, CO
17640/6	14.0	11.9	114.7	44.9	21.2		3.0	6200		El Paso Cnty, CO
17808/10	19.0	14.6	117.2	45	23	97	1.0	4000	.24	Boulder, CO
11779/1	24.0	23.7	102.0	45	26		0.4	1800		Denver, CO
9215/22	8.0	13.6	111.1	45.9	25.4	87.5	3.5	4100	.29	Douglas Cnty, CO
16547/1	88.0	15.3	112.7	46	27		1.2	7000		Denver, CO
17438/11	14.0	19.8	107.4	47.7	24.1	89		3000	.27	Aurora, CO
17755/4	4.0	9.6	108.9	48	32	81	2.0	2400	.40	Jefferson Cnty, CO

Table A-2 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYSTONES)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
17488/7	29.0	17.7	110.4	49	28	94	0.7	3200	.30	Arapahoe Cnty, CO
16488/6	19.0	20.1	106.5	49	24	91	0.6	3200	.26	Arapahoe Cnty, CO
17438/4	7.0	21.9	103.7	49.6	27.1	91		5500	.30	Aurora, CO
16474/9	9.0	19.4	107.5	51	25	97	1.0	2400	.26	Grand Junction, CO
17673/4	4.0	28.8	90.6	51	13	75		3800	.17	Arapahoe Cnty, CO
17808/3	14.0	16.3	115.0	51	27	100	2.9	10000	.27	Boulder, CO
16488/9	19.0	20.3	106.9	52	31	94	1.1	5400	.33	Arapahoe Cnty, CO
17647/8	8.0	22.3	109.5	52	21		1.9	4000		Littleton, CO
16590/1	3.0	20.1	106.2	52	31		1.5	4500		Arapahoe Cnty, CO
9191/2	8.0	23.0	104.7	52.8	27.1	98.8	0.9	2250	.27	Colorado Sprgs, CO
16474/1	7.0	13.2	117.1	53	32	98	5.5	12000	.33	Grand Junction, CO
17647/13	9.0	21.7	100.6	53	25		1.4	4300		Littleton, CO
16473/2	9.0	16.6	110.2	54	34	68	1.5	5900	.50	Aurora, CO
16393/1	9.0	16.4	111.6	54	41	93	3.8	15000	.44	Thornton, CO
16547/4	61.0	20.5	106.2	54	26		1.2	8000		Denver, CO
17602/9	14.0	19.9	107.2	54	25	98	1.4	3500	.26	Douglas Cnty, CO
17403/5	18.0	24.1	92.1	54.8	32.1		0.9	3300		Arapahoe Cnty, CO
16817/11	8.0	16.4	112.8	55	35	95	1.2	4600	.37	Westminster, CO
17431A/4/4	12.0	15.5	109.8	55	36	98	4.8	7400	.37	Thornton, CO
17450/8	25.0	24.5	100.2	55	32	88		3500	.36	Golden, CO
17533/1	19.0	20.0	106.3	55	33	94	3.0	5300	.35	Broomfield, CO
17647/11	3.0	20.3	91.7	55	27	92	4.6	15000	.29	Littleton, CO
17614/2	12.0	15.4	108.5	55	33.1	80	5.3	20000	.41	Colorado Sprgs, CO

Table A-2 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYSTONES)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTICITY INDEX (%)	-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
16590/3	19.0	23.8	97.4	56	25		0.8	3000		Arapahoe Cnty, CO
17450/7	7.5	24.9	99.2	57	25			4200		Golden, CO
16596/9	3.0	18.4	99.8	57.2	30.7	75	9.2	18000	.41	Lamar, CO
17437L5184	13.0	20.9	102.7	58	39	98		11000	.40	Cherry Creek, CO
17601/1	19.0	23.1	101.1	59	31	99	0.9	2300	.31	Douglas Cnty, CO
17607/1	19.0	16.9	110.6	59	40		1.9	2900		Westminster, CO
17836/3	18.0	25.6	97.4	60	29	76	3.2	9100	.38	Denver, CO
17405/13/6	7.0	14.1	101.0	61	39	93	3.8	6700	.42	Bismarck, N.D.
17673/4	4.0	31.3	88.1	62	30	83		7000	.36	Arapahoe Cnty, CO
17405/9/5	5.0	21.2	100.0	61	41	95	1.1	2700	.43	Bismarck, N.D.
11799/20	14.0	36.4	85.7	63	30		1.5	4400		Denver, CO
17604/2	8.0	20.2	105.2	65	39	99	3.1	8000	.39	Lincoln Cnty, CO
17630/1	8.0	29.1	78.3	65	29			1400		Lakewood, CO
16615/B1CS2	9.0	33.1	81.5	66	37	97	1.1	3200	.38	California
11799/25	24.0	36.2	85.1	66	28		0.8	2400		Denver, CO
17614/1	4.0	15.4	112.0	67.9	46.7	92	7.4	17000	.51	Colorado Sprgs, CO
16735/23	24.0	37.1	81.1	68	34		0.3	1500		Jefferson Cnty, CO
16615/B1CS5	24.0	38.4	80.7	71	31	90	1.1	4100	.34	California
16615/B4CS2	14.0	34.0	83.1	72	40	91	2.3	21000	.44	California
17576/4	7.0	24.3	99.8	72	47	94	4.7	20000	.50	Englewood, CO
17673/1	3.0	30.7	90.8	72	36			6500		Arapahoe Cnty, CO
11799/6	24.0	38.5	81.1	73	45		0.5	2000		Denver, CO
9215/5	8.0	25.3	97.2	76.5	50.6	95.4	2.1	3600	.53	Douglas Cnty, CO

Table A-2 (continued)

SUMMARY OF LABORATORY TEST RESULTS (CLAYSTONES)

HOLE	DEPTH (FEET)	NATURAL MOISTURE (%)	NATURAL DRY DENSITY (PCF)	ATTERBERG LIMITS		-200 SIEVE (%)	(%) SWELL	SWELLING PRESSURE (PSF)	PI/-200	LOCATION
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)					
16615/BICS4	19.0	35.6	80.5	77	45	100	1.1	3000	.45	California

APPENDIX B

COMPARISON OF MEASURED VERSUS PREDICTED SWELLING PRESSURES

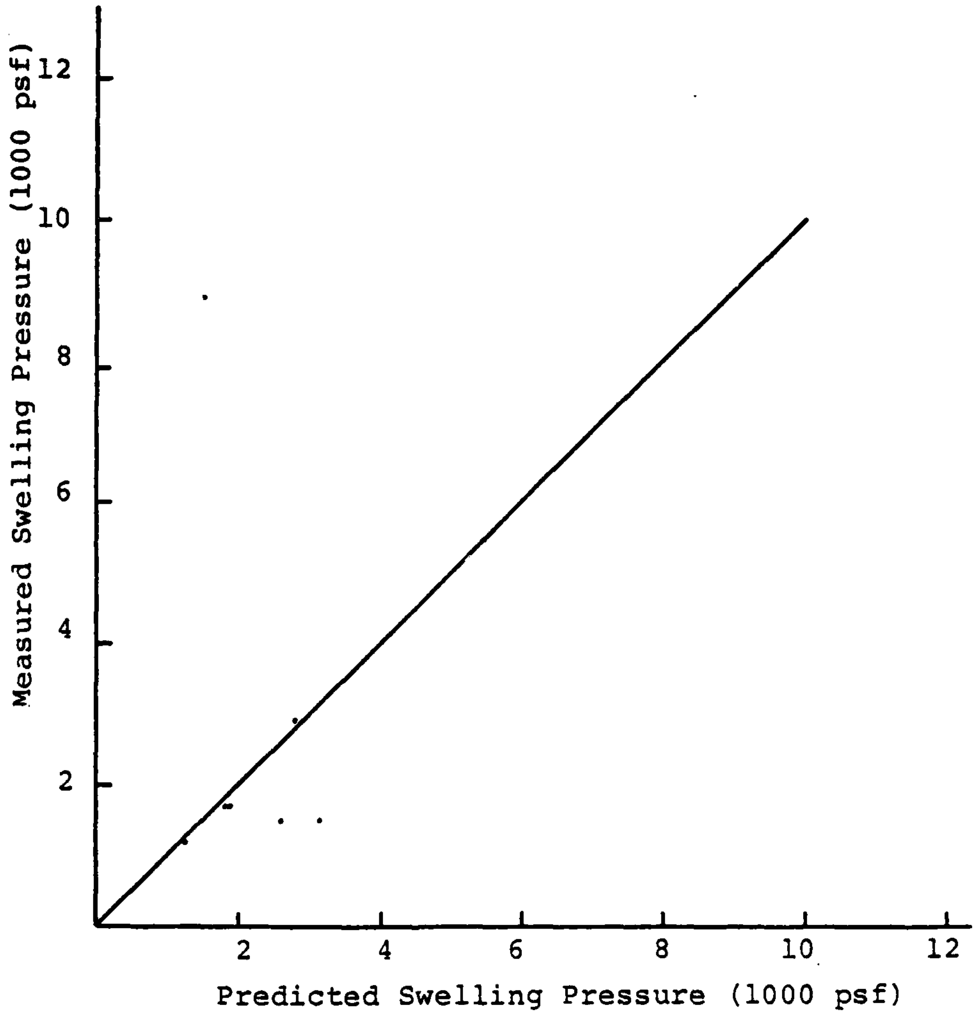

Figure B-1

Measured Versus Predicted Swelling Pressures for Claystones (Liquid Limit Range: 32-39)

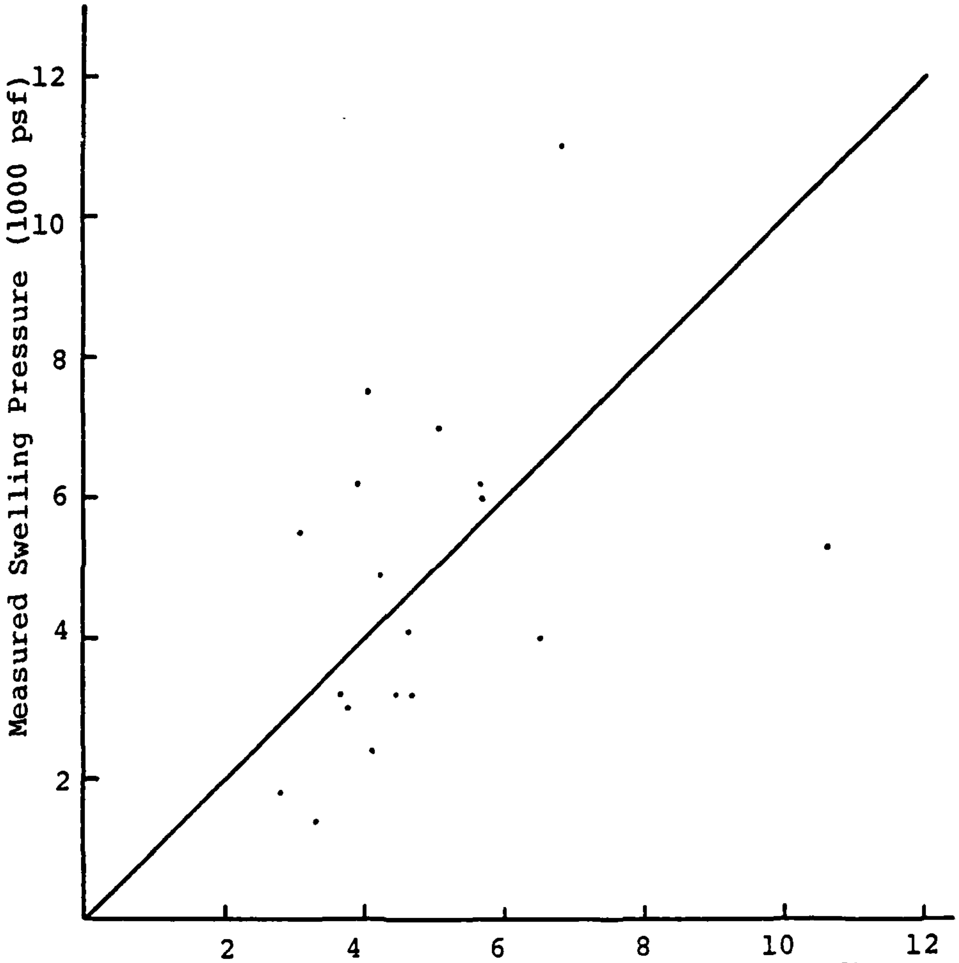

Figure B-2

Measured Versus Predicted Swelling Pressures for Claystones (Liquid Limit Range: 40.7-49.6)

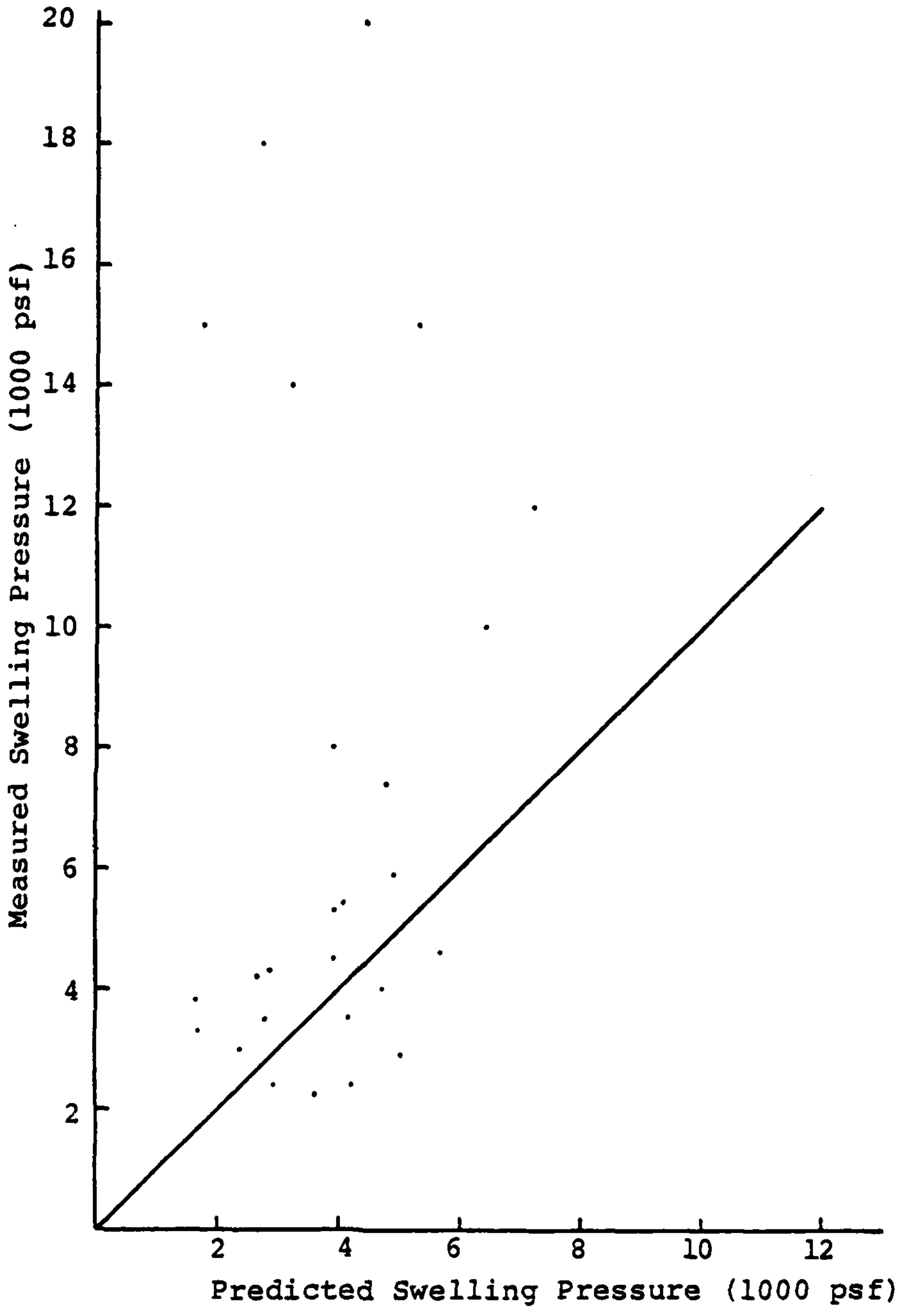

Figure B-3

Measured Versus Predicted Swelling Pressures for Claystones (Liquid Limit Range: 51-59)

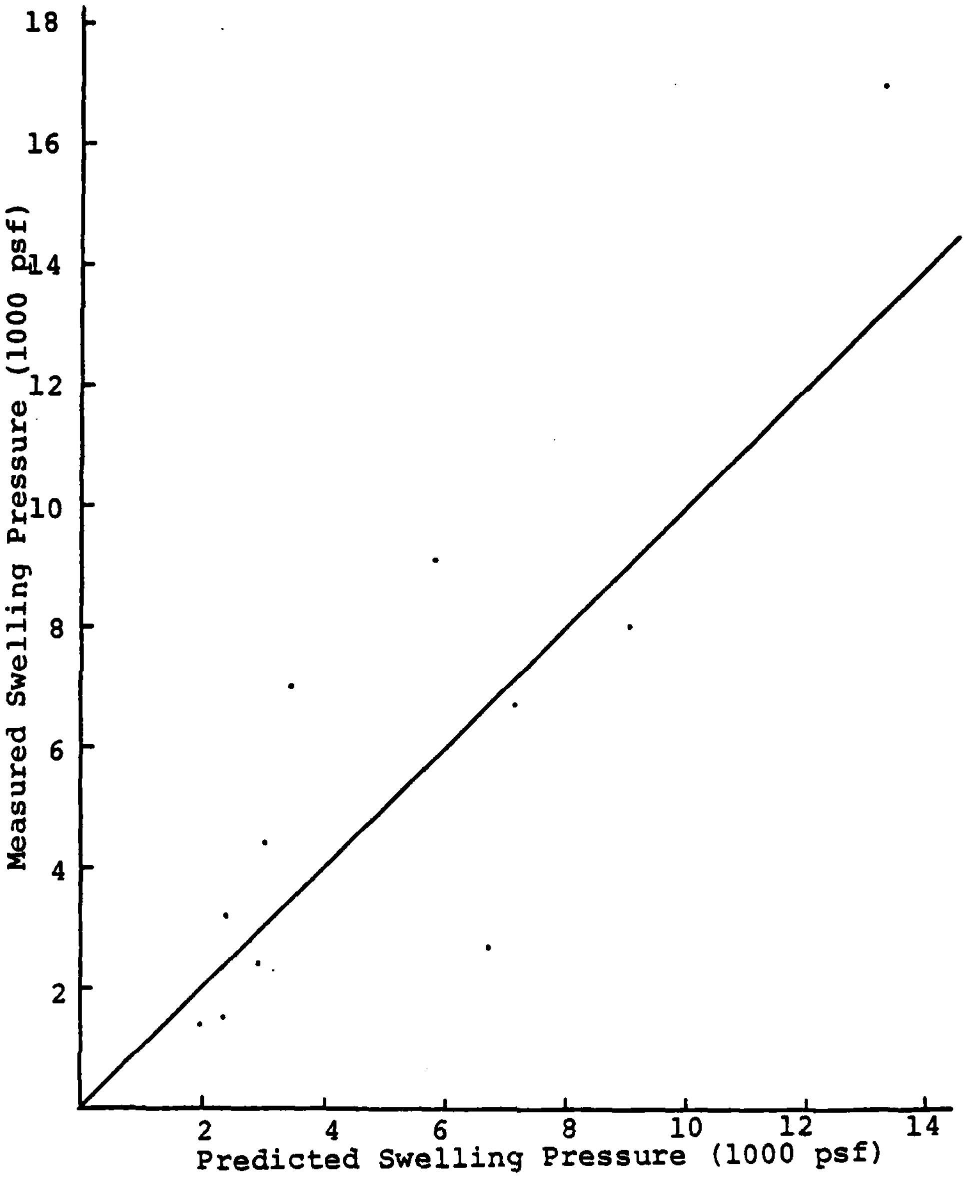

Figure B-4

Measured Versus Predicted Swelling Pressures
for Claystones (Liquid Limit Range: 60-68)

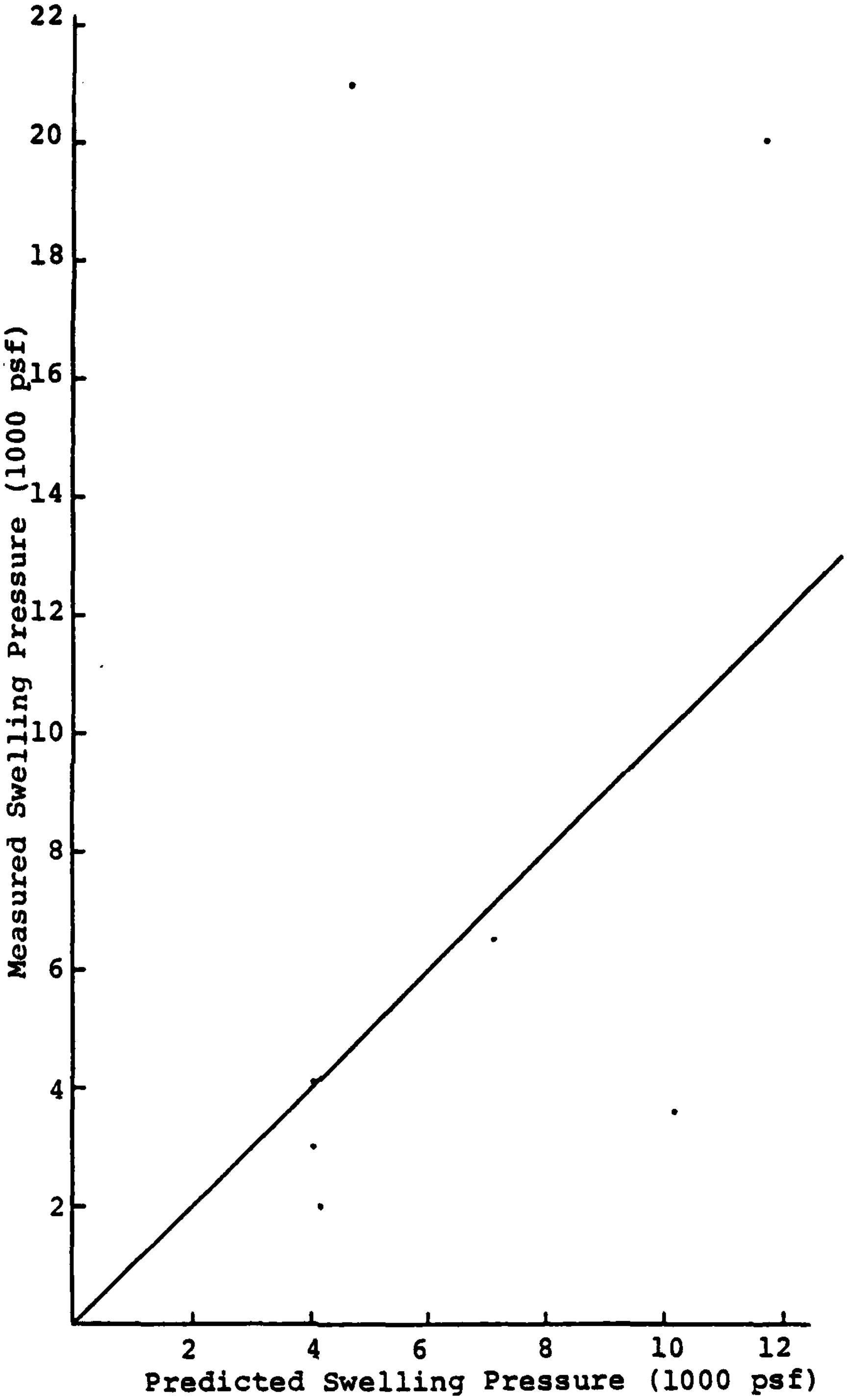

Figure B-5

Measured Versus Predicted Swelling Pressures
for Claystones (Liquid Limit Range: 71-77)

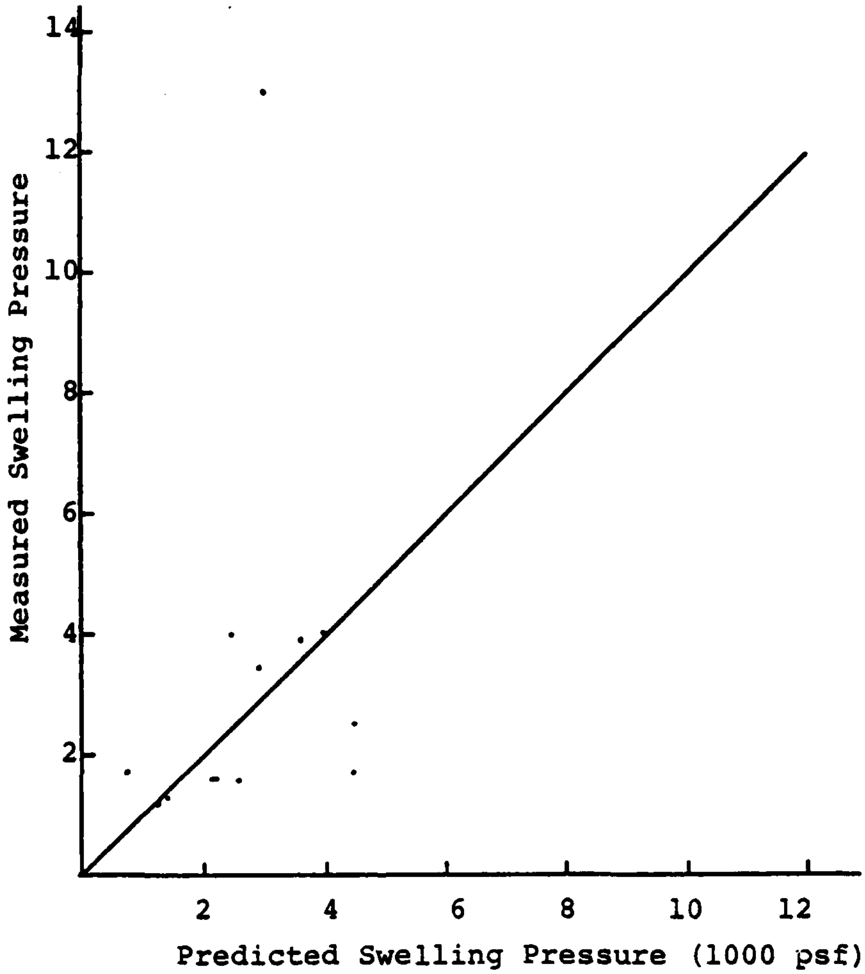

Figure B-6

Measured Versus Predicted Swelling Pressures for Clays (Liquid Limit Range: 25-29.9)

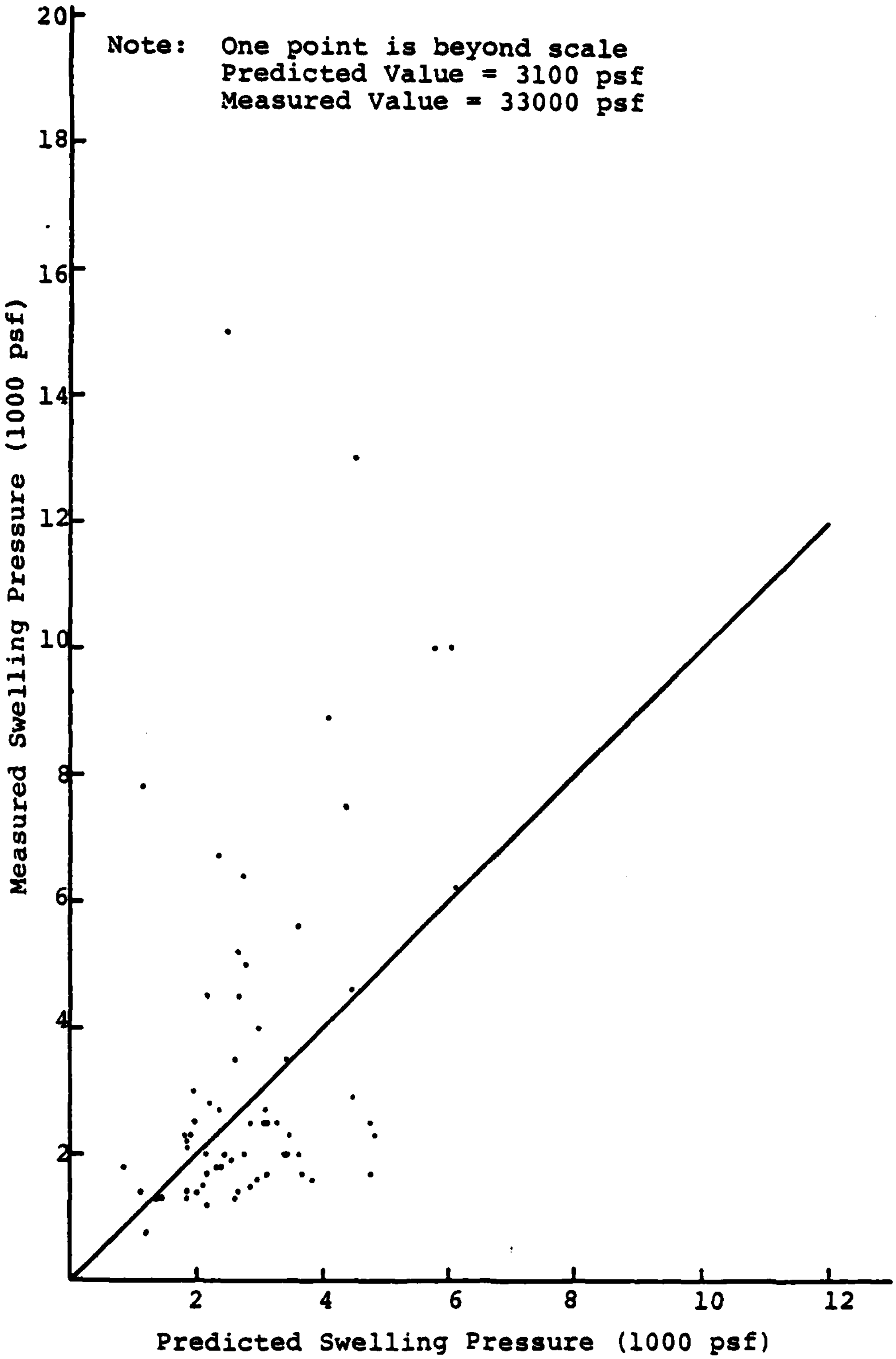

Figure B-7

Measured Versus Predicted Swelling Pressures for Clays (Liquid Limit Range: 30-39.9)

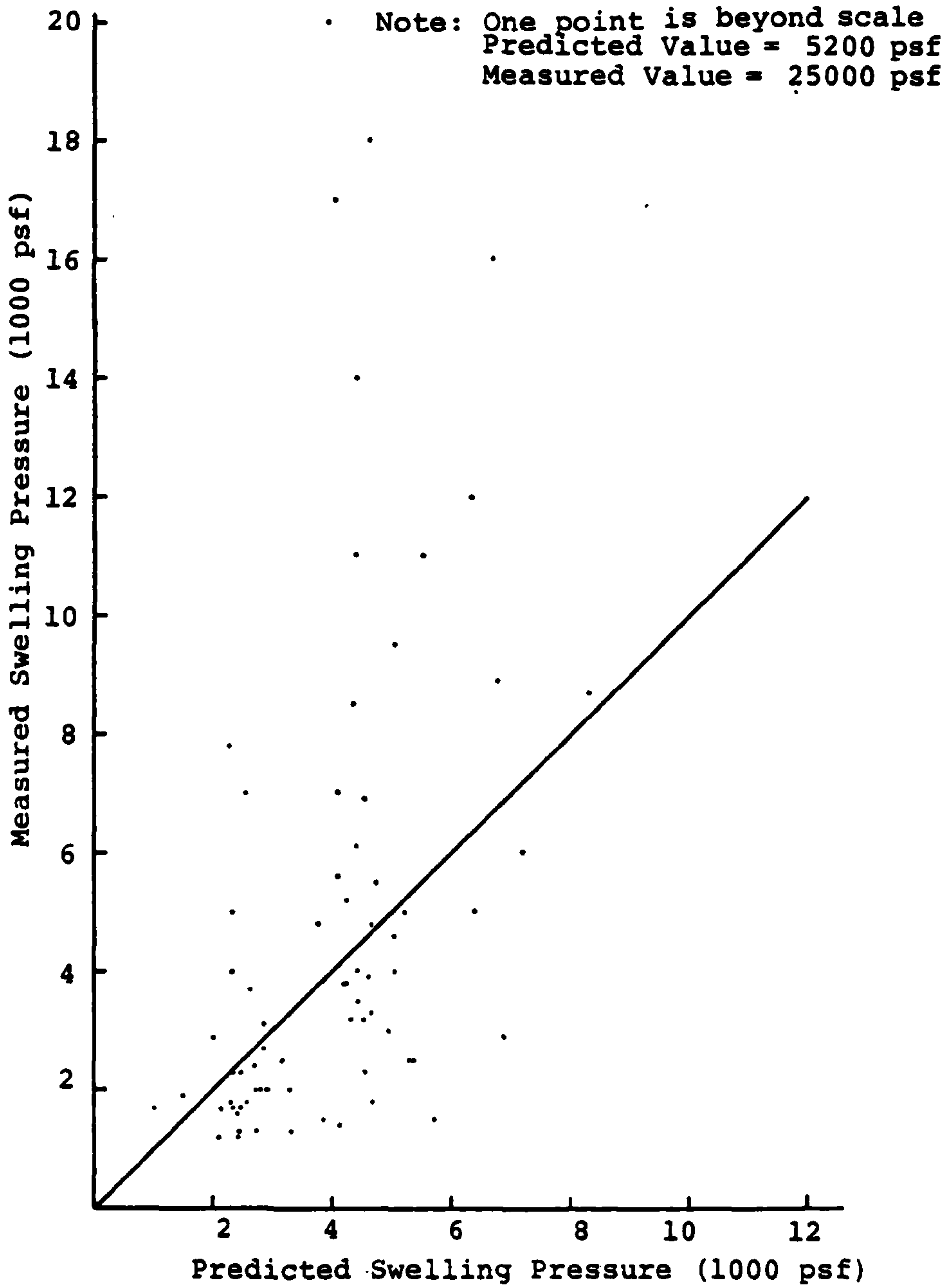

Figure B-8

Measured Versus Predicted Swelling Pressures for Clays (Liquid Limit Range: 40-49.5)

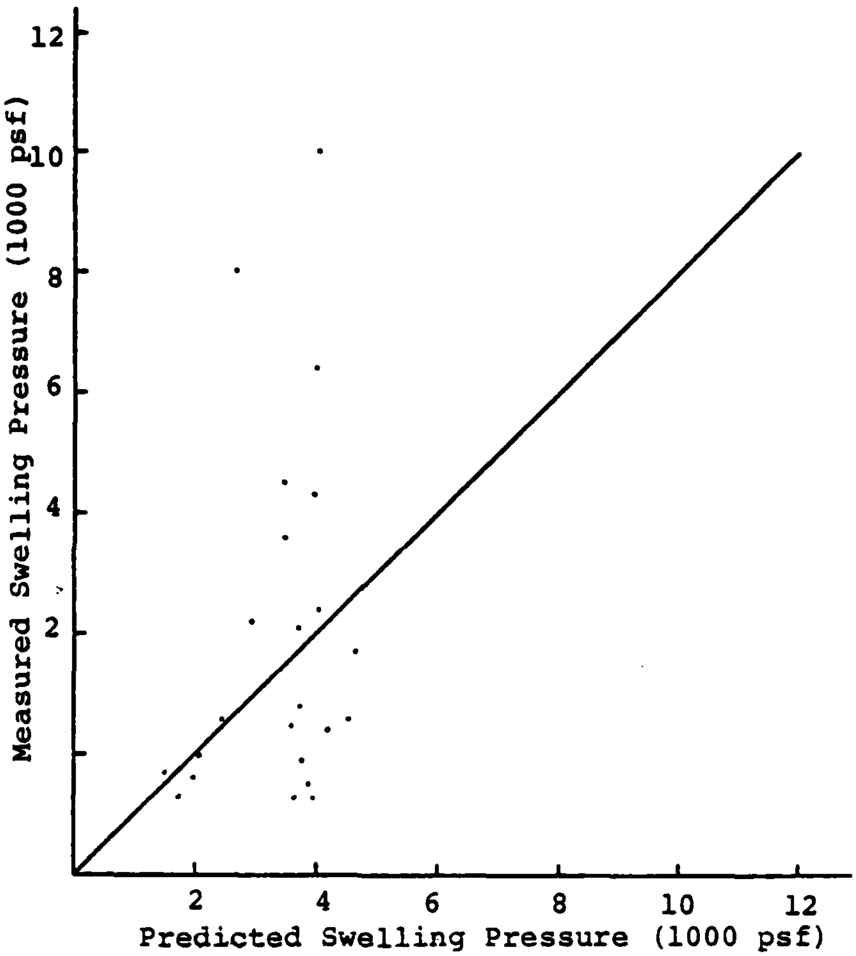

Figure B-9

Measured Versus Predicted Swelling Pressures for Clays (Liquid Limit Range: 50-58)

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